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of the

AMERICAN CONCRETE

INSTITUTE

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with the direction of the PUBLICATIONS COMMITTEE: W. A. SLATER, chairman; DUFF A. ABRAMS, P. H. BATES, S. C. HOLLISTER, A. R. LORD, A. N. TALBOT, BENJAMIN WILK, HARVEY WHIPPLE, Secretary.

Papers: Original papers, furthering the objects of the society, are invited. They should be addressed to the Secretary, and their consideration by the Publications Committee will be facilitated if: (1) a brief but comprehensive outline or summary precedes or at least accompanies the completed paper; (2) manuscript and photographs are available in duplicate or triplicate; (3) original tracings or other drawings, from which copies are readily made, are available.

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JOURNAL
of the
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INSTITUTE

News Letter

Tentative Program

26TH ANNUAL CONVENTION
NEW ORLEANS
FEBRUARY 11, 12, 13

(The Institute meetings are preceded by the convention of the National Concrete Products Association and by several important Institute committee meetings, Monday, Feb. 10. The Cast Stone Institute program is in progress the same week. To accommodate the numerous early arrivals Institute registration will begin Monday morning—top floor, Roosevelt Hotel.)

MIXING AND PLACING CONCRETE

2 p. m. Tuesday, Feb. 11

CONSTRUCTION SPECIFICATION FOR CONCRETE WORK ON ORDINARY BUILDINGS reported by Committee 502, Mixing and Placing Concrete on Buildings, Arthur R. Lord, author-chairman. This is no ordinary specification as those who have read the November JOURNAL know. In leading the convention discussion Mr. Lord will be prepared for attacks on what some believe are radical specification requirements.

CONSTRUCTION SPECIFICATION FOR FABRICATING AND SETTING REINFORCING STEEL reported by Committee 503, W. F. Zabriskie, author-chairman. As this JOURNAL goes to press the committee is yet to reach perfect accord. If the accord is reached in time the specification will appear in

The Railroads granted "fare and a half" round trip rates on the "Certificate Plan." When you buy your railroad ticket ask for a "certificate." 150 such certificates properly validated at the A. C. I. registration desk will insure half fare return. This is a considerable item to many of our members. You can help them to save it by obtaining a certificate —whether *you* want it or not.

the February JOURNAL, with addenda under the title "A Steel Setter's Primer." Mr. Zabriskie wrote the latter to reach with the fundamentals the man who actually places the steel and it looks like a valuable supplement to the specification proper.

CONCRETING METHODS AT CHUTE A CARON DAM, (well up on the fringe of things on the Saguenay river) is a paper full of interest for every man designing and placing concrete. I. E. Burks, concrete technician for the Alcoa Power Co., is writing the paper. The Institute's committee, 801, Durability of Concrete, A. E. Lindau chairman, went up to see how Burks proposes to get durability—(a thousand yards of it a day) and found out; but prefers that Mr. Burks tell the story.

SPECIFICATION FOR CENTRALLY MIXED CONCRETE, will be reported by Committee 504, Miles N. Clair, author-chairman. If we are going to buy our concrete by the truckload and have it dumped ready-to-use on the job, water-ratio and all, as thousands of yards are being bought, rules must be set up for a confirmed confidence in this new enterprise.

DESIGN AND OPERATION OF CENTRAL MIXING PLANTS, was the assignment of Committee 602, Frank I. Ginsberg, chairman. The committee's first report will be in brief papers by members of his committee who may be expected to answer Committee 504 by telling how a specification may be met readily by the rigidly controlled conditions of the central mixing plant.

AFTER THE CONCRETE IS MIXED AND PLACED—CURING

8 p. m. Tuesday, Feb. 11

METHODS OF CURING CONCRETE by H. F. Gonnerman, Manager Research Laboratory, Portland Cement Association. The results of an extensive investigation.

WINTER CONCRETING METHODS, reported by Committee 604, R. C. Johnson, author-chairman, as the basis of a "recommended practice." Discussion is looked for from members of the committee. New Orleans will provide ideal conditions for discussing sub-zero practice.

VARIATIONS IN STANDARD PORTLAND CEMENTS, reported in the November JOURNAL by Committee 202, P. H. Bates, author-chairman, will be up for discussion.

PRELIMINARY STUDIES OF HIGH PRESSURE STEAM-CURING IN THE MANUFACTURE OF CONCRETE MASONRY UNITS, by P. M. Woodworth, Assistant Engineer, Research Laboratory, Portland Cement Association. Discussion will be lead by Ray C. Kiser, manager Crume Brick Co., Dayton, Ohio, in whose plant the investigation was made.

CONCRETE PRODUCTS

9:30 a. m. Wednesday, Feb. 12

A STUDY OF VOLUME CHANGES IN CONCRETE MASONRY WALLS, (preliminary results of an important investigation) by W. D. M. Allan, manager Cement Products Bureau, Portland Cement Association.

RECOMMENDED PRACTICE FOR THE MANUFACTURE OF STANDARD CONCRETE MASONRY UNITS—a preliminary study by Committee 708,

P. M. Woodworth, chairman, of a revised standard, based on modern practice and present knowledge of essentials.

RECOMMENDED PRACTICE FOR THE USE OF CAST STONE, reported by Committee 704, Louis A. Falco, chairman; presented for discussion and early publication in the JOURNAL as proposed practice to be presented for tentative adoption in 1931.

COLOR IN CONCRETE PRODUCTS, a summary of good practice, by Raymond Wilson, author-chairman Committee 703.

PLANT DESIGN FOR SINGLE OR MULTIPLE SHIFT OPERATION, as reported by Benjamin Wilk, author-chairman Committee 707, and discussed by critic members of the committee. This report should provide an opening wedge for a general discussion of plant design and operating problems.

Wednesday afternoon, Feb. 12

Exploring "Old Man River" on the Mississippi river steamer Capitol—for everybody who thinks it's worth a dollar.

REINFORCED CONCRETE BUILDING DESIGN—FLOOR FINISH

8 p. m. Wednesday, Feb. 12

THE REINFORCED CONCRETE COLUMN INVESTIGATION—(Its possible influence on design) by Committee 105, W. A. Slater, chairman.

ECONOMICS OF TALL BUILDING DESIGN reported by R. R. Zipprodt, author-chairman Committee 405.

DEFLECTIONS OF REINFORCED CONCRETE MEMBERS, a progress report by T. D. Mylrea, author-chairman, Committee 307.

GOOD PRACTICE IN FINISHING FLOORS reported by Committee 802, Portland Cement Floor Finish, John G. Ahlers, chairman. Stereoptican views of floor cross sections showing the results of methods of finish and why some floors fail—presented with brief discussion by W. E. Hart, secretary of the committee.

INSTITUTE BUSINESS—FLOOD PROTECTION

1:30 p. m. Thursday, Feb. 13

BUSINESS OF THE INSTITUTE (including installation of new officers and directors).

THE PRESIDENT'S ADDRESS by E. D. Boyer.

DESIGN AND CONSTRUCTION OF THE BONNET CARRE SPILLWAY (to divert Mississippi flood waters around New Orleans through Lake Pontchartrain) by Capt. Helmer Swenholt, Corps of Engineers, U. S. A.

MAKING AND PLACING CONCRETE REVETMENT MAT (for which interesting plant and procedure have been developed) by Lieut. Morris W. Gilland, Corps of Engineers, U. S. A.

THE DESIGN OF GULF SHORE PROTECTION STRUCTURES by J. B. Converse, consulting engineer, Mobile, Ala.

CEMENTING OIL WELLS (far from being a fully developed and understood technique) will be the subject of a paper by J. E. Hough, Portland Cement Association. The Program Committee explored numerous likely sources of information on this sub-

ject—they all led back to Mr. Hough. He in turn makes the subject doubly interesting by intimating that it is still full of dark mystery with a very great deal still to learn.

THE DINNER

7 p. m. Thursday, Feb. 13

The post prandial program will test the emotional elasticity with its extremes of honoring the distinguished medalists (in three presentations, see elsewhere this NEWS LETTER) to a half hour or so of special New Orleans entertainment of a distinctive character—but that is to be experienced, not announced.

HOTEL RESERVATIONS

IN ARRANGING for its convention in New Orleans the Institute was assured that while February is a heavy tourist month everyone will be accommodated if he uses reasonable forethought in making reservations.

The Roosevelt—the headquarters hotel—is setting aside for American Concrete Institute members subject to advance reservations, the following:

	Per Day
50 Single rooms and bath at	$5.00
50 Single rooms and bath at	6.00
100 Twin bed rooms and bath at and at the Bienville hotel (same management—bus service to and from the Roosevelt).	10.00
40 Single rooms and bath at	4.00

Other hotels where tentative reservations have been made assure A. C. I. members accommodations as follows:

ST. CHARLES HOTEL

25 Single rooms with bath at	$4.00 and $5.00
100 Double rooms with bath at	7.00 and 9.00

HOTEL MONTELEONE

25 Single rooms with bath at	$4.00
20 Rooms with double bed and bath	7.00
30 Rooms with twin beds and bath	8.00

JUNG HOTEL

50 Single rooms with bath at	$4.00
25 Double rooms with bath at	6.00
25 Double rooms with bath (twin beds) at	7.00

IN NEW ORLEANS WEEK OF FEBRUARY 10

MEMBERS of the Institute will find that the convention headquarters in New Orleans is close by many of the historically and romantically interesting spots of the old city. Memories of pirates, priests and princes are interwoven with historic grounds and buildings of the Vieux Carre.

The old Absinthe House was a famous wine room from 1826 to the Volstead enactment, and before that, the building is said to have been the headquarters of the pirate brothers, Pierre and Jean Lafitte, where they kept a blacksmith shop as a side line. The Cabildo, Chartres Street, was built in 1795, and was used as a government building early in the nineteenth century when Louisiana was transferred to the United States. It is now a museum. The history of the Cathedral occupying an entire square goes back to 1794. In Congo Square, early in the 1800's, African slaves danced their Voodoo rites. The local recommendation is that the French Market should be seen about midnight; built under Spanish regime about 1791, it is distinctly an old world scene. An interesting tea room occupies the building which was once the Louisiana Bank, and until his death in 1884, the home of Paul Morphy, chess king. Place D'Armes was a public square and drill ground laid out in 1720, and was the scene of the transfer of Louisiana by France to Spain in 1769, and of Louisiana to United States in 1803, now known as Jackson Square. The Pontalba Buildings are interesting architecturally and historically, as apartments erected in 1850 by Baroness de Pontalba, in one of which Jenny Lind lived in 1851. St. Anthony's

Garden in the rear of the Cathedral was a favorite dueling ground. St. Louis Cemetery was the burial ground for St. Louis Cathedral before 1800. The graves, above ground show many interesting names—Charles LaSalle (brother of the explorer), Dominick You (Lieutenant of the Pirates Lafitte) and Paul Morphy among them.

Not to tempt the members too much away from the convention, it nevertheless seems only fair to mention that the horse-racing season is in full swing in New Orleans. There are some very famous restaurants and—if you must, night clubs.

ROUND TRIP FOR A FARE AND A HALF

THE JOURNEY to the annual convention is necessarily longer this year for the average member but because New Orleans is one of the most interesting cities of the United States (someone has said the third most interesting) and because its climate in February is friendly, it is likely that many members of the Institute will make the convention the occasion of a holiday.

The usual arrangements have been made with the railroads for reduced fare, when the going and coming journeys are made over identical routes. The round trip fare will be a fare and a half, saving 25% on the mileage cost. This is dependent upon the proper validation of 150 certificates deposited at the registration desk at the convention. Whether you personally are in a position to take advantage of the reduced rate or not, be sure in every case to ask for a certificate when you buy your going ticket. The validation of 150 certificates will mean a substantial saving to many Institute members. The reduction in fare is made on account of the combined attendance at the conventions of the American Concrete Institute and the National Concrete Products Association, and the reduced fare is available to members and dependent members of their families. **Be sure to ask the ticket seller for a certificate when you buy your going-ticket.**

TO MAKE YOU WELCOME

THE NEW ORLEANS CONVENTION COMMITTEE is able, interested, broadly representative and very hospitable. Its members will do all they can to make you glad you attended the 26th Annual A. C. I. Convention. Here's the "Who's Who" of the committee:

JOHN L. PORTER, (chairman), director of the water purification stations, Sewage and Water Board, New Orleans.

SCOTT THOMPSON, (vice-chairman), vice-president and manager of the Lone Star Cement Co., Louisiana, subsidiary of International Cement Corporation, operating numerous plants in this and other countries.

W. R. MACATEE, (secretary), district engineer of the Portland Cement Association.

JOHN F. COLEMAN, consulting engineer, New Orleans, recently named for next president, American Society of Civil Engineers.

W. B. DAVEY, acting chief engineer of the Orleans Levee Board—constructing and maintaining levees in and about the city.

PROF. DONALD DERICKSON head of the School of Civil Engineering of the College of Engineering, Tulane University, New Orleans.

JOHN T. EASTWOOD, principal assistant engineer of the Sewerage and Water Board, New Orleans. This board has charge of sewers (sanitary and storm drainage) and water systems of the city. Mr. Eastwood has direct charge of the sanitary sewers.

SAMUEL FOULKES, JR., of the Convention and Tourist Bureau of New Orleans.

MAJOR R. F. FOWLER, district engineer of the first New Orleans River District, with headquarters in New Orleans River District, looking after navigational facilities, particularly in various passes in the lower Mississippi river, also of the Intercoastal Canal.

F. H. FOX, associate professor, school Engineering, Tulane University, New Orleans.

PROF. W. B. GREGORY, head of the school of Mechanical and Electrical Engineering of the College of Engineering, Tulane University. Has done much research work of recent importance as Consulting Engineer of the Mississippi River Commission, in connection with the experiments conducted looking to adoption of a design for the Bonnet Carre Spillway, a part of the Mississippi River flood protection work.

W. O. GRIFFEN, superintendent, U. S. Independent Alcohol Co., New Orleans.

GEORGE R. HAMMETT, A. M. Lockett & Co., New Orleans, contracting mechanical engineers.

HARRY B. HINDEPLITE, state highway engineer, of the Louisiana State Highway Department at Baton Rouge, Louisiana, actively engaged at this time in concrete road construction program.

MAJOR W. H. HOLCOMBE, district Engineer, War Department, U. S. Engineer Office, New Orleans, Louisiana, executive of the New Orleans River District, in charge of all river work between a point just below Vicksburg, and the mouth of the Mississippi River.

HARRY F. JACOBS, chief engineer of the Board of State Engineers, with headquarters in New Orleans. This Board has to do with state flood protection measures, and the construction and maintenance of a system of levees within the state.

WALTER F. JAHNCKE, secretary-treasurer of Jahncke Service, Inc., (building materials) and vice-president and treasurer of Jahncke Dry Docks, Inc., (brother of Assistant Secretary of the Navy, Ernest Lee Jahncke).

JOHN KLORER, commissioner of Public Property, Commission Council, City of New Orleans; past president of the American Society for Municipal Improvements, and chief engineer-elect of the New Orleans Levee Board, responsible for levee protection about the city.

WALTER MOSES, treasurer of the Louisiana Engineering Society.

F. A. MUTH, secretary of the Louisiana Engineering Society, and engineer U. S. Lighthouse Service.

OLE K. OLSEN, dealer in building specialities, structural and reinforcing steel, contractors equipment.

JOHN REISS, New Orleans contractor.

R. B. ROESSLE, Jones, Roessle, Olschner & Weiner, New Orleans and Shreveport, Louisiana architects.

SOLIS SEIFERTH, Weiss, Dreyfous & Seiferth, Inc., New Orleans architects.

DR. H. S. SHILSTONE, head Shilstone Testing Laboratory, inspecting engineers, New Orleans.

SAMUEL YOUNG, chief engineer, Board of Commissioners, Port of New Orleans in charge of all dock construction.

BRYSON VALLAS, city engineer of New Orleans.

ARCHITECTURAL CONCRETE INTERIORS AND ACOUSTICS

FROM A LETTER BY DONALD B. PARKINSON*

We have used concrete architecturally, on complete exteriors with very satisfactory results. A few examples are the Elks Club at Santa Barbara, the Chesterfield building on Wilshire Boulevard, Los Angeles, the Packard Sales and Service building in Hollywood. In each of these all of the architectural features were cast in with the structure of the building in plain concrete, the desired finish color being obtained by a very thin dash and skim coat of stucco. This finish coat is so thin that it still leaves visible the joints in the form work and does not destroy the vigorous structural appearance of the concrete.

We have used concrete much more extensively as an architectural medium in interiors, chiefly in ceilings, but for entire rooms as well. Probably the most important example being the ceiling of the Council Chamber in the Los Angeles City Hall, on which job we were associated with John C. Austin and A. C. Martin. In this particular room the ceiling is designed in beam and girder construction with the elements carefully proportioned and detailed from both an architectural and structural angle. The ceiling was finished in very rich decoration directly on the concrete, the result being very handsome and distinctive.

In every such use of concrete in a decorative way the results have been entirely satisfactory so far as appearance goes. The only difficulty we have experienced is in overcoming bad acoustic conditions where a large proportion of the area is in exposed concrete. This, of course, can be corrected by the use of acoustic materials but the cost involved is often quite high. We believe it would be very helpful to Architects if your Institute could develop concrete surfaces which would have acoustic qualities.

*John Parkinson and Donald B. Parkinson, Architects, Los Angeles.

THE HONOR ROLL TO JANUARY 8

W. F. WAY'S activities on the west coast place him at the top of the Honor Roll on January 8—three weeks to go before the final 1930 tally is made.

I. E. Burks, between batches on a big dam job high up on the Saguenay river, and Harry Gonnerman, extracting data on curing concrete at the P. C. A. laboratory—have managed a tie score for second place.

Messrs. Emerson, Fenkell and Upson tie in third place, the two latter both at a disadvantage—absent for several months in Japan.

Messrs. Ahlers, Bragger and McMillan are in fourth place but John Ahlers doesn't propose to stay there. We happen to know he is making an eleventh month drive in New York City along with completing New York's quota on the Column Test Fund.

Then there are nine members with 3 each and an even dozen who may before February 1, get the one more

member they need to score on the Honor Roll.

The top few—just how many has still to be decided—will win special awards at convention time.

Here is the list:

W. F. Way	**10**
I. E. Burks	**9**
H. F. Gonnerman	**9**
H. B. Emerson	**6**
George E. Fenkell	**6**
M. M. Upson	**6**
J. G. Ahlers	**4**
E. Y. Bragger	**4**
F. R. McMillan	**4**
Duff A. Abrams	**3**
W. D. M. Allan	**3**
L. E. Andrews	**3**
G. J. Eyrick, Jr.	**3**
H. J. Gilkey	**3**
A. R. Lord	**3**
Alan Moncrieff (India)	**3**
Hale Sutherland	**3**
Miguel Villa (Cuba)	**3**

AND THOSE BELOW NEED TO SCORE AGAIN

F. C. Brookhouse	2
J. Campbell & Sons, Ltd.	2
B. E. Clark	2
A. K. Dandekar	2
E. W. Dienhart	2
T. Campbell Gray	2
J. C. Hawkins	2
W. S. Loucheim	2
B. A. Rich	2
F. E. Richart	2
C. J. Seibert	2
John Tucker, Jr.	2

When you buy your rail road ticket for New Orleans ask for a Certificate

AWARD OF MEDALS

ANNOUNCEMENT is made by the Board of Direction of the award of medals, to be presented at the Institute's 26th Annual Dinner, New Orleans, the evening of February 13, as follows:

To L. G. Lenhardt, assistant engineer in charge of land tunnels, division of engineering, Department of Water Supply, Detroit, the Wason medal "for the most meritorious paper presented at the 1929 convention" under the title "The Concrete Lining of Detroit Water Tunnels."

To Harry F. Gonnerman and P. M. Woodworth, respectively manager and assistant engineer, Research Laboratory, Portland Cement Association, Chicago, Wason medals for Research, for the work reported in their 1929 joint paper "Tests of Retempered Concrete."

The Wason medals were founded by Leonard C. Wason, Boston, past president of the Institute, in 1916. They are of bronze and are distinguished recognition of valuable contributions to the special knowledge of the Society's field of technical interest.

To Prof. F. E. Turneaure, Dean, College of Mechanics and Engineering, University of Wisconsin, Madison, the Henry C. Turner gold medal, founded 1928 by Henry C. Turner, New York, past president of the Institute, and presented not oftener than once each year for "Notable achievement in or Service to the Concrete Industry." The medal is accompanied by a certificate setting forth the nature of the service or achievement—which in the award to Professor Turneaure is "for distinguished service in formulating sound principles of reinforced concrete design."

IN CONSTANT REPAIR

COMMITMENT—BY THE AUTHOR

Dr. Samuel Johnson, of dictionary fame, has said, "A man, Sir, must keep his friendships in a state of constant repair." It is no less true of one's stores of information than of one's friends that they diminish rapidly through inattention and lack of replacement. Perfectly canned facts of 1925 are discovered to have spoiled by 1929 and they should not be left upon the shelves to give a false impression of abundance. It is my purpose, *if the Editor condescends to permit me*, to seek public assistance in renovating and refurnishing the storehouse of my mind. That, it seems to me, is a logical benefit that membership in the American Concrete Institute should confer and I propose to claim it, if others in the Institute will be so agreeable as to assist in the work of "constant repair." Such assistance implies an obligation of further print, in which the new and shiny information received shall be passed on to fellow members. This I agree to—always assuming an indulgent Editor and a restrained blue pencil. It would involve a series of frank revelations of the unfortunate state of my mental stores and of later proud displays of newly acquired wealth. Should this beginning article,

"1. THE FIFTH INGREDIENT,"

appear without a general title and without a numeral, you may judge that the Editor looks upon this proposal with suspicion and hostility,* and that another budding author has suffered the usual *coup de grace*.

*Either suspicion or hostility might have been enough. "Serials" are under a cloud of editorial skepticism. A fanfare in January may be a mere squeak by June. But Mr. Lord has revealed not merely an intention but submitted some future "copy." He has some things to say sufficiently provocative that we shall rely not only upon Mr. Lord, but upon you who read for the perpetuation of this corner of the *News Letter*. Surely, Mr. Lord is not alone in the desire to repair a slightly damaged stock of information. The Editor will be glad to maintain a clearing house for repair parts.—EDITOR

1. THE FIFTH INGREDIENT

BY ARTHUR R. LORD

I MUST admit at the very start that I am one of the Institute's wisest men on the subject of admixtures, for I realize well how very little is positively known about this mysterious subject. I have been forced to think on it. The Board of Direction has delegated to me, in part, the task of thinking on several subjects which they are too lazy or too wise to hunt down collectively. I encounter admixtures at every turn, in the technical press, on the job. They have money and they move in the best concrete society. Their representatives have even climbed to the rather inaccessible heights where I do daily battle with printed and personal salesmen. They have caused me amusement, concern, unpleasant indecision.

But I have discovered a way to dispose of them. Strange berries in the woods have ever intrigued me to taste and see if they are good—and if promptly spit out, they do no harm. You may know little or nothing about their real qualities, but you have satisfied, in a degree, your scientific curiosity. It is so with admixtures. Is that not, indeed, a fair picture of the way most of us treat them with our trifling "job experiments" for which a long suffering admixture manufacturer furnishes a few hundred pounds of this or that for us to play with? It deadens our distrust, substitutes a feeling of familiarity, flatters our intellect, and gets us nowhere. Clever salesmen are good psychologists and play upon our human limitations. Occasionally we wake up and ask for facts instead of playthings, but not often nor too insistently.

I have had amusement. On one occasion 'Admixture J (as my cautious scientific friends label it to our great

profit and enlightment) was strongly recommended and the contractor was impressively "sold." Always eager for new knowledge, we made all arrangements for a job test—a "practical" demonstration as contrasted with the childish ways in which professors go about such things in laboratories. The interested parties were all assembled and a card was sent to the mixer captain with instructions to return it with the first cart of concrete containing the new material. In a few minutes it arrived and the debate began. The architect's superintendent was hostile and saw no benefits—only increased segregation. The contractor noted a marked increase in "workability", as did the salesman. I acquired a generous coat of cement freckles trying to follow up an impression that the concrete did appear "fatter." Then I happened to glance at the card which a workman had handed to me, and read: "Will have to get a can to measure the dope—this will take ten or fifteen minutes. Fred."

On another occasion, working with the well known Admixture Q (and this is *not* a typical instance of my dealings with this manufacturer) we concreted a typical girder with rather congested reinforcement. The placing was done with care—neither more nor less effort than usual. The slump was maintained at our usual average value. As the operation was completed, I was called to the telephone. A brief message, combined with a ride on the hoist both ways, brought me back very quickly. I found a sunny African spading that particular girder for dear life—or rather for a consideration of one dollar cash in hand! This admixture resulted in a very superior surface on this girder. The salesman was considerably provoked when I insisted, without further explanation, that his method was altogether too expensive for me to recommend to my client.

Even in my correspondence this subject has embarrassed me. In a letter to an eminent Colonel in Washington, I inadvertently referred to admixture E as of volcanic origin. He devoted a full page to disabusing me of this erroneous notion, describing in minute detail the life history, early struggles, and catyclismic end of the diatom family. All this, I now maintain, I already knew. I have myself seen the beds near Lompoc, California, where incalculable numbers of these sea animals were laid to eternal rest, until so rudely disturbed by my friend Wig. The Colonel was sufficiently punished, for three days later he met a violent end in an automobile accident.

Most of my acquaintance with admixtures has been disturbing rather than amusing. If they would only act *convincingly*, I could be happy either with or without them! Why do so many people paint their characters in so different colors? Would that there were not so many of them. The time was when we had but one or two to consider. On Wacker Drive we used but one—hydrated lime—and that but part of the time. The balance of the time we used extra cement. For that work hydrated lime and cement were the only admixtures without political backing, a necessary qualification. One compound, even the name of which is now passed from my memory, had the highest aldermanic recommendation—recommendation is not the word—*insistence*. It was guaranteed to produce the most superior strength if used in accordance with the printed instructions. And it would—for one of the instructions limited the water to 3.5 gallons per sack of cement! That was

too stiff even for an assembly of "moppers-up."

Now for my lament, my appeal for information, my questions of which no five-year-old has a more copious supply —and perhaps no more unanswerable. How shall we be saved from our indecisions and uncertainties?

I wish to know more about Celite—much more. Also about Bentonite, Barnsdall Admix, Aquagel, Pumicite, Intertol, Gar Kem, Omicron, clay (both colloidal and ordinary), hydrated lime, ammonium stearate, and many more that escape my memory at the moment. Having been pestered by numerous questionnaires, I prepared a particularly keen one of my own for admixture salesmen and manufacturers exclusively. Many have been taken away from my office—*one* returned. Here is the reason.

QUESTIONNAIRE FOR ADMIXTURE SALESMEN

Name of Admixture.....

1. Statement of nature of material.
   Liquid, powder, premixed with cement at mill, chemical composition (if public).
   Variation in product of different manufacturers, selling under same trade name.
   Source of the material.
2. Purposes for which used and method of application.
3. Theory of its effectiveness.
4. Cost to produce and to apply.
5. Properties of the material.
   a. weight, voids, specific gravity, fineness and gradation.
   b. moisture carried and bulking effects.
   c. how measured on job.
   d. chemical reaction on other materials likely to be used with it.
   e. deterioration due to age or storage.
   f. cautions (hot water dangers, etc.)
   g. will it be carried away by percolating water?
6. Effect on Properties of Concrete (including effect of variation in product of different manufacturers).
   a. weight.
   b. workability and segregation.
   c. strength (and w/c-relation) at different ages and under different curing conditions.
   d. permeability and absorption.
   e. shrinkage and volume changes including plastic yield.
   f. fire resistance and fireproofing value.
   g. color and texture and ease of finishing in various effects.
   h. modulus of elastic and modulus of rupture.
   i. accoustical and insulating value.
   j. wear, dusting, abrasion.
7. Economy claimed by direct use of material.
8. Economy claimed as secondary result (or superior result claimed).
   a. due to saving in dead weight.
   b. due to saving of patching.
   c. due to omission of water proofing.
   d. due to reduced size of members.
   e. due to omission of plastering or other finish.
   f. due to internal curing by water held in pores, etc.
9. Design aids in case new tables or diagrams are needed.
   a. for use of high strength concrete, etc.
   b. different value of E.c
   c. new column design diagram, etc.
10. Construction Difficulties.
    a. will it "gum up" the mixer.
    b. practical "kinks" for field use.

I should be happy to receive answers to any of my questions on any admixture from anybody not engaged in its manufacture or distribution, as well as from those who are. Have you used some admixture with satisfaction? Under what conditions? What do you know about its "plasticizing" value, its soluble alkali content, its efficiency as compared with added cement at the same cost? What tests have you had made and how were all the variables controlled and determined, so that the meaning of the tests may be known? There seemed to be some difference of opinion in the discussion of the Abrams paper on admixtures at Atlantic City as to what constitutes equal "workability" and doubtless this same question will arise over the use of slump measurements in the illuminating paper by McMillan and Lyse, on curing and watertightness, in the December JOURNAL.

A relatively large amount of test data is held in the private files of corporations, architects and engineers,

It would be gratifying indeed, if a considerable portion of this hidden material might be brought to light, combined, studied and reported in a summary as a result of this inquiry. It seems quite certain, however, that extensive new research will be required to resolve our various doubts or to answer my imposing questionnaire at all adequately. There are signs that such research may soon be undertaken—possibly by a single manufacturer at first—and that we shall know better, in the not too distant future, where we stand on at least one of our many proposed fifth ingredients.

REINFORCED CONCRETE COLUMN INVESTIGATION

FURTHER WORD FROM COMMITTEE 105*

DECEMBER 13 and 14, Committee 105, Reinforced Concrete Column Investigation, held two full-day and evening sessions at which many details of the proposed tests were threshed out and the preliminary outline was substantially completed. With the completion of this outline, the scope of the proposed study can be stated somewhat more fully than was possible in the former announcement. As reported elsewhere in this issue, the funds for the investigation are developing in a very gratifying manner. It is desired to obtain $20,000 over and above the indirect contributions being made by the two Universities—(University of Illinois and Lehigh University) where the tests will be carried out. A conservative estimate of the direct and indirect contributions of these two laboratories places their value at $15,000.

The general purpose of the column study is to establish the fundamental relationships between the load, elastic properties, and ultimate strength as affected by such factors as quality of concrete, amount and quality of longitudinal and lateral reinforcement, rate and method of applying load, effect of continuous loads over long periods of time, and size of columns. This is a very comprehensive program and requires a large number of test columns, designed to bring out successively the influence of each factor to be studied, for various combinations of the remaining factors. One of the first questions to be settled is the method of applying the load to the test column. Only a section of a column shaft can be conveniently introduced into the testing machine. While this can be made truly representative of a portion of a column as to amount and arrangement of reinforcement, some difficulties are encountered in attempting to bring the load upon the column in the manner that prevails in the structure. The first group of tests includes a small number of columns having different arrangements of end details. The purpose being to find a suitable end condition that best simulates the action of transfer of column loads in a building. The columns for subsequent tests will be designed in the manner shown to be most suitable by the results from this group.

Another group of tests of a more or less preliminary character, but much more extensive than the first group, is designed to show how the elastic properties and ultimate strength are affected by the rate of applying load to the column. This series includes four different strengths of concrete, three grades and four percentages of longitudinal reinforcing steel, two per-

*See November *News Letter* for previous statement.

centages and two grades of spiral steel. Two methods of loading will be employed. One of these is the method generally used, that is, loading rapidly, and the other is one that involves a slower application of the load up to the ultimate strength of the column.

A third group of tests is designed to determine the effect of long-time loading at ordinary working stresses. Two sets of columns are included, one to be tested after curing in moist air, and one after curing in dry air. Each set includes parallel columns to be stored under the same conditions, but carrying no load. At the end of one year, the loaded and unloaded columns will be tested for ultimate strength in the manner determined from the results of the first two groups. In this series, three kinds of concrete are included, and three percentages of both spirals and vertical reinforcement.

A fourth group of columns, somewhat similar to the above, will be made to study the effect of a continuously applied load at intensities approaching the ultimate strength of the column.

In the four groups of tests just described, which number over 500 columns, the test columns will be 8 in. in diameter and 60 in. long. To relate the results more closely to columns encountered in every day practice, two groups of columns are planned using core diameters of 12, 20, and 28 in. with and without the 2-in. shell for fireproofing. These will include two kinds of concrete and two combinations of spiral and vertical reinforcement. This group of large columns will extend the range to the limit of capacity of the three-million-pound testing machine now being installed at the University of Illinois.

Work on this investigation will be pushed as rapidly as the program will permit. It is expected that the first group of columns will be completed in January, and the columns for the long-time tests started in February.

The desirability of including tests of columns with eccentric loading has been recognized, but the conclusion has been reached that this should follow the completion of the program here outlined.

CONTRIBUTIONS RECEIVED FOR REINFORCED CONCRETE COLUMN TEST FUND TO JAN. 10.

New York City Local Committee

Received since Dec. 11

Ley & Co., Fred T.	**$100.00**
Lockwood Green Engineers, Inc.	**37.50**
McGraw & Co., F. H.	**100.00**
Ransome Concrete Machinery Co., Dunellen, N. J.	**100.00**
Snare Corp., Frederick	**100.00**
	437.50

Acknowledged Dec. Journal

Barney-Ahlers Constr. Corp.	250.00
Ferguson Co., John W., Paterson, N. J.	250.00
Foundation Co., The	100.00
Goodwin-Gallagher Sand & Gravel Corp.	100.00
Industrial Engineering Co.	100.00
MacArthur Concrete Pile Corp.	100.00
McCormack Sand Co., Wm. J.	100 00
Nassau Sand & Gravel Co.	100.00

New York Trap Rock Corp.	100.00
O'Brien Brothers, Inc.	100.00
Rosoff Sand & Gravel Corp.	100.00
Spencer, White & Prentis, Inc.	100.00
Steers, Inc., Henry	100.00
Stewart & Co., Inc., James	250.00
Turner Construction Co	100.00
White Construction Co.	200.00
	$2,150.00
New York City Total to Jan. 10, 1930	**$2,587.50**

Cincinnati Local Committee

Acknowledged Dec. Journal

Avril Tru-Batch Concrete, Inc.	$25.00
American Aggregates Corp.	25.00
Cincinnati Builders Supply	15.00
Dugan, J. N.	10.00
Euphrat & Hanly	25.00
Ferro Concrete Const. Co.	100.00
Fisher-DeVore Const. Co.	50.00
Hall Co., T. J.	20.00
Hannaford & Sons, Samuel	25.00
Hodges Const. Co.	50.00
Miller & Son, Wm.	5.00
Moores-Coney Corp.	15.00
Nutting Co., H. C.	25.00
Penker Construction Co.	25.00
Van Camp Stone Co.	25.00
Cincinnati Total to Dec. 10, 1930	**$ 440.00**

MONTREAL

Anglin-Norcross, Ltd.	**25.00**
Canada Cement Co.	**200.00**
Peck Rolling Mills	**100.00**
Steel Company of Canada, Ltd.	**100.00**
	$ 425.00

WAUKESHA, WIS.

Butler Bin Co.	**150.00**

SAN FRANCISCO

Transit Mixers, Inc.	**50.00**

Acknowledged Dec. Journal

CHICAGO

Turner Construction Co.	100.00

BOSTON

Turner Construction Co.	100.00

PHILADELPHIA

Turner Construction Co.	100.00

SPECIAL CONTRIBUTIONS

Concrete Reinforcing Steel Institute	$2,500.00
Rail Steel Bar Association	$2,500.00
Total Cash Received to Jan. 10, 1930	**$8,952.50**

DURABILITY OF CONCRETE

THE INSTITUTE'S Committee 801, Durability of Concrete, A. E. Lindau, chairman; F. R. McMillan, secretary; has been an active committee in the last year, although it has no report for presentation at the convention. The committee has made two important inspection trips, one in western New York, one in extensive survey of many structures in Canada, and it plans immediately following the Institute convention in New Orleans (where the committee will meet) to view a number of gulf coast structures and other concrete work a little further north. Out of this work it is tentatively planned to produce a number of individual reports for JOURNAL publication.

New Orleans has many visitors in February—make your hotel reservations early

ASSEMBLING PORCELAIN INSULATORS WITH PORTLAND CEMENT

A LETTER FROM J. C. PEARSON

Mr. Pearson, besides being the Institute's Director for the third district, solves abstruse technical problems for the Lehigh Portland Cement Co. In an unguarded moment a few days ago he casually mentioned one of these problems in a letter to the secretary. The editor scans all the secretary's mail and persuaded Mr. Pearson to write a somewhat longer letter that would tell more about one of the uses of cement that, to most of us, is not an every-day matter.—EDITOR

ONE of the comparatively rare uses of portland cement in neat form occurs in the manufacture of porcelain insulators for high tension electric power lines. Not only are the metal pins and caps, which support the insulators, cemented to the porcelain, but large insulators are frequently built up by the assembly of a number of porcelain parts, all rigidly cemented together. In general it may be said that no other cementing material has been found, in any way comparable on a cost basis, that serves the purpose as well as portland cement.

Nevertheless there have been failures of insulators, particularly in the so-called pin type, in which the upper section, supported only by the cement joint which attaches it to the section below, has cracked and separated into two approximately equal portions, as if split by an expansion of the cement joint between the two sections. Since this phenomenon can be most obviously and directly explained by expansion of the hardened cement, and since it is generally known that hardened cement will expand slowly when exposed to moisture, the theory that the failures are caused by the cement has been generally accepted by electrical engineers.

Investigations have shown, however, that many of the failures cannot be attributed directly to the cement, in the sense that the cement itself is not at fault, but rather that lack of something in the design of the insulator, or in the mixing, placing and curing of the cement paste is the true source of trouble. It is evident at once that volume changes of some sort are the immedate cause of this type of failure, but an attempt to diagnose any given case brings one up against the difficulty that essential facts in the history of the case are not known.

Let us assume that the cement is under suspicion, as it usually is when the cause of defective concrete work is not obvious. One inquires whether anything was known about its volume change characteristics, whether it was subjected to any sort of soundness tests, or if there was any knowledge of its composition. Almost invariably nothing of this sort is known, and yet it *should* be known before any blame can be fairly placed on the cement. One then inquires whether the location was such that severe freezing might occur after a thaw or after a storm, and if the answer is yes, a more probable explanation of the trouble is in sight, particularly so if the cement paste had been mixed with an excess of water, or if for any reason there might have been relatively high shrinkage in the joint at the time of assembly. There is no doubt that many insulator failures have been due to expansion in freezing, and no doubt that this danger can be practically eliminated by attention to

the design of the insulator and by proper processing in the assembly.

Another type of rapid volume change failure is possible where freezing temperatures are the exception or do not exist at all. Let us assume that the cement joints are reasonably tight in an insulator exposed directly to the sun's rays on a hot, humid, summer day. The temperature of the insulator may under such circumstances be well above 100° F. Suddenly a storm arises and a heavy downpour of cold rain, perhaps only a few degrees above the freezing point, strikes the hot insulator; the upper shell undergoes a quick contraction, and although the coefficient of porcelain is less than that of cement, a disruptive stress will have been set up in the porcelain before the temperature of the joint has fallen sufficiently to relieve it.

Thus it is seen that there are at least three possible causes of insulator failure from purely mechanical sources:

1. The possibility of abnormal expansion in the cement, which can be eliminated by selection.

2. The possibility of expansion by freezing.

3. The possibility of rapid differential shrinkage.

These two may be greatly minimized, as stated above, by rational design and correct processing. The additional effect of electrical stresses in contributing to insulator failures is not well known, but it is probably safe to assume that the uncertainty in this respect is largely involved in the lack of knowledge of mechanical stresses, and that both of these must receive primary consideration in the design of the insulator.

DISCUSSION

Discussion of papers and reports published this month should be available before April 1, with a view to their publication in the JOURNAL for May, 1930. It is recommended that prospective contributors to discussion make their work available early. Late discussion or the later contributions to discussion which may be too voluminous for a single issue, will be carried over possibly for several months.

From the point of view of the service that the cement must render, there are two main questions that are not yet answered in regard to the cement itself. The first is: does steam curing, now commonly in use by most insulator manufacturers, result in a net advantage to the producer, all things considered? The obvious advantage lies in the rapid acceleration in strength and the limited amount of curing space required, but against this is the cost of equipment and the more serious question whether the steam cured insulators are equal in quality to those cured under normal temperatures? The second question is: Is the possibility of growth or slow expansion in a hardened neat cement paste of good quality from absorption of moisture a factor that may limit the useful life of cemented insulators? A long period of carefully conducted experiments will be necessary to furnish the answers to these questions, but so far as information is now available, properly selected cement, and proper processing in the assembly of insulator parts, promise

the elimination of the majority of mechanical troubles which are now more or less a source of worry in certain phases of the electrical industry.

NEW MEMBERS

Applications for membership in December, 1929, approved by the Board of Direction are:

Bates, Harold W., 429 Griswold St., Detroit, Mich.

Bon, N. M., The Concrete Assoc. of India, Mull Bldg., Outram Road, Lucknow, U. P., India.

California Talc Co., Inc., 837 Jackson St., Los Angeles, Calif. (Geo. L. Ratcliffe, Pres.)

Cone, R. G., c/o Tacony-Palmyra Bridge Co., Palmyra, N. J.

Consolidated Cement Corp., 111 West Monroe St., Chicago, Ill. (A. W. King.)

Epstein, A., 2001 W. Pershing Road, Chicago, Ill.

Florida State Road Dept., Testing Division, Gainesville, Fla. (H. C. Weathers.)

Gillette, Harold S., 1813 Fort Worth Nat'l. Bank Bldg., Fort Worth, Texas.

Hastrup, Harold K., 1350-B Argyle St., Chicago, Ill.

Hagy, Ernest A., Ferro Concrete Construction Co., Cincinnati, Ohio.

Holt, C. D., 412 Husband St., Stillwater, Okla.

Hurd, W. F., 3777 No. Meridian St., Indianapolis, Ind.

Jones, R. C., 233 Balboa Heights, Canal Zone.

Kynnersley, T. R. S., c/o Concrete Association of India, E-2 Clive St., Calcutta, India.

Loeser, Jesse J., 157 West 63rd St., New York, N. Y.

Lavoie & Delisle, Chicoutimi, Que., Canada.

Maeser, Mieth, Mass. Institute of Technology, Room 1-212, Cambridge, Mass.

Martin, Juan San, Box 630, Havana, Cuba.

Merrill, David H., 2006 Pelham Ave., Los Angeles, Calif., Pacific Coast Bldg. Conference.

Nishkian, L. H., 525 Market St., San Francisco, Calif.

Orelite Co., Inc., 109 Broad St., New York, N. Y. (Samuel Janovici, Pres.)

Paquette, J. E., Box No. 8, Kenogami, Que., Canada., Alcoa Power Co.

Pettus, L. A., Hydraulic Press Brick Co., 705 Olive St., Room 1000, St. Louis, Mo.

Schlottenmeier, Joseph F., 394 Brunswick Ave., Trenton, N. J.

Schulzke, Wm. H., Peoples Bank Bldg., Moline, Ill.

Thomas, Harry E., P. G. & E. Laboratory, 4245 Hollis St., Emeryville, Calif.

Toronto Ready Mix Concrete, Ltd., Room 1017—159 Bay St., Toronto, Ont., Canada.

Steele, R. F., Alcoa Power Company, Kenogami, Que., Canada.

Uhrich, M. J., c/o Henry & McFee, Northern Life Bldg., Seattle, Wash.

Watson Company, P. O. Box 1002, Dallas, Texas. (G. W. Anderson.)

Wood, James, 26 W. 83rd St., New York, N. Y.

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Moment and Shear Diagrams for Continuous Beams and Rigid Building Frames

BY NORMAN M. STINEMAN*

The four diagrams of moment and shear curves for continuous beams and the fourteen diagrams pertaining to rigid building frames which accompany this paper as Diagrams 1 to 18, inclusive, were originally prepared in the spring and summer of 1929 for the Sub-Committee on Reinforced Concrete of the Committee on Materials, Loads and Stresses appointed by the Merchants' Association of New York to prepare a new building code for that city. The action of the Merchants' Association was undertaken upon the invitation of the mayor of the city. For convenience, the sub-committee just mentioned, of which the author of this paper was a member, will hereinafter be designated as the New York Sub-Committee on Reinforced Concrete, or the New York Sub-Committee.

The Tentative Building Regulations for Reinforced Concrete issued by the American Concrete Institute under date of May, 1928, were utilized as the working basis for the sub-committee's report. However, after extended discussion of the moment coefficients specified in Sections 708 and 709 of the American Concrete Institute regulations, the committee felt that designers who wish to compute the actual moments and shears determined from the principles of mechanics should not be hampered by arbitrary coefficients.

Engineering literature, both in this country and in Europe, contains many highly technical papers on the analysis of continuous beams and rigid frames. As a rule, however, the work has fallen far short of being made intelligible to the average structural designer. The members of the New York Sub-Com-

*Editor of *Concrete*, 400 W. Madison St., Chicago: formerly Structural Engineer, Portland Cement Association.

mittee on Reinforced Concrete realized fully the importance of employing more exact methods of analysis than those ordinarily followed in the design of the structural frames of buildings; but they realized equally well the necessity for having such information available in convenient and usable form. If the general run of structural designers, more especially those engaged in designing the structural frames of buildings, are to be induced to make more accurate analyses of structures they must be given reasonably accurate design data in more convenient form than has been given them heretofore.

A modification of the foregoing paragraph may be made with respect to continuous beams, for which tabulations and diagrams have been published in forms that are quite convenient to use. The trouble has been that in many cases the authors limited their calculations to three spans. Several French and German handbooks include values of bending moments in continuous beams up to five or six spans.

The foreign-language handbooks and textbooks obtained or consulted by the author contain little of value on the subject of rigid building frames. American textbooks include considerable data on such frames; but here the designer is again confronted by the fact that too much preliminary work is left to him. The average structural designer is dismayed, and with good reason, when he is confronted by complicated diagrams and curves involving the use of a number of variables which are not clear to him. If he can determine the meaning of such variables and the method of using the diagrams and curves only by careful study of many pages of text of a highly technical nature, he will in most cases not undertake the task. The truth is that in most designing offices he would not be permitted to take the time necessary to follow the calculations through, even if he were inclined to do so. The final result is that he falls back on the more convenient, though inaccurate, bending moment coefficients specified in the city building code.

DISTINCTION BETWEEN CONTINUOUS BEAM AND RIGID FRAME

There is an unfortunate lack of information regarding the dividing line between a continuous beam and a rigid frame. The designer is expected to use his judgment in the matter; but in most cases the regulations in the city building code decide the

matter for him by making no distinction. Most building codes specify arbitrary moment coefficients for continuous beams and girders, and apply those same coefficients to beams and girders which frame into columns.

The New York Sub-Committee on Reinforced Concrete has provided a definite and logical dividing line. A line of beams framing into other beams or girders at the same point but on opposite sides shall be considered and designed as continuous beams. Beams or girders framing into columns and built to act integrally with such columns shall be considered and designed as parts of rigid frames. In the special case where the girders run in one direction only, with small joists framing directly into the girders, the joists which frame into the columns would, under this definition, constitute a part of a rigid frame rather than a continuous beam. For practical reasons, however, a joist so located may be reinforced in the same manner as the remaining joists, which under this definition have been designed as continuous beams. The designer will be on the safe side in doing this.

Following the distinction made between continuous beams and rigid frames, the New York Sub-Committee report then continues with a provision to the effect that the positive and negative bending moment and shear coefficients at any section and the location of the points of inflection in slabs, beams, girders and columns, whether freely supported, continuous or restrained, shall be determined on the basis of recognized principles of mechanics for those combinations of loading which will cause maximum stress at the section.

The provision just quoted was amplified in the New York Sub-Committee report by a note recommending that the moment and shear charts accompanying this paper be accepted as indicating standard practice in this respect.

FLOOR SLABS OF SHORT SPAN

The only deviation made by the New York Sub-Committee was in reference to ordinary floor slabs framing into beams or girders at one or both edges, and whose span is not in excess of eight feet. For such floor slabs framing into beams or girders at both edges the committee specifies a center moment of $wL^2/16$ and a negative moment of $wL^2/12$ at the supports. Where such slabs are freely supported at one edge, the positive moment near

the center of the span is assumed at $wL^2/12$ and the negative moment at the continuous edge is assumed at $wL^2/10$. The distance from the face of a continuous support to the point of inflection in these floor slabs is assumed at 1/5 of the span length.

Arbitrary coefficients for floor slabs of spans not exceeding eight feet can be justified from the fact that the bending moment in short slabs is not great, and the further fact that the beams and girders which support the slabs in many cases are large enough to create a condition approaching complete fixity. The moment coefficients selected for this special case are conservative.

METHODS EMPLOYED IN COMPUTING VALUES FOR DIAGRAMS

While the methods employed in computing the moment and shear values for the diagrams need not enter into an understanding of the use of the diagrams, an explanation of the computations will nevertheless be given. The following notations and definitions will be of interest:

A *continuous beam* is a beam framing into other beams or girders and extending over two or more spans.

A *rigid building frame* consists of a system of girders and columns in which the girders and columns are built to act integrally with each other.

The *stiffness factor* of a column, girder or other structural member is the numerical value obtained by dividing the moment of inertia of the member, expressed in inches to the fourth power, by its free length, expressed in inches.

Column stiffness and *girder stiffness* are terms employed to denote the degree of stiffness represented numerically by the "stiffness factor."

The *free length* of a column is the length between the floor level and the cap of a column in a flat-slab system, or between the floor level and the under side of the lowest intersecting girder above, extending in the direction in which moments are being considered. The *free length* of a girder is the length between faces of supports, or between supporting brackets.

w = Uniformly distributed dead load and live load combined, in lb. per lin. ft., with the live load placed to produce the greatest moment or shear at the point in question. When the dead load alone is considered, w is the dead load in lb. per lin. ft.

L = Span length in feet, measured from face to face of supports. In connection with the computation of the stiffness factor of a girder, L is the free length of the girder and is expressed in inches.

H = Free length of a column or strut, expressed in inches when its moment of inertia is expressed in inches to the fourth power.

I = Moment of inertia of a girder, column or other structural member expressed in inches to the fourth power. In the case of a reinforced concrete structural member the "transformed" section of the steel must be taken into account.

$\frac{I}{L}$ = The "stiffness factor" of a beam or girder.

$\frac{I}{H}$ = The "stiffness factor" of a column or strut, or similar structural member.

COMPUTATIONS FOR CONTINUOUS BEAM DIAGRAMS

So far as the continuous beam curves are concerned the work was comparatively simple. While most text-books dealing with continuous beams stop with three spans, moment and shear values for continuous beams of four and five equal spans were obtainable from French and German handbooks. Only a moderate amount of original work was necessary to prepare the bending moment curves.

For the shear values at the mid-point of continuous beams, however, considerable work was required because of the absence of published data covering this special case. For illustration, consider the case of the maximum positive and negative shears at the mid-point of the third span of a continuous beam of five equal spans. The maximum positive shear at the center of the span is obtained by placing live loads on the second and fifth spans and on the right half of the third span. The maximum negative shear at the same point is obtained by loading the first and fourth spans and the left half of the third span. The shear values at the mid-points of four and five equal spans were obtained only after making a complete analysis.

COMPUTATIONS FOR DIAGRAMS FOR RIGID BUILDING FRAMES

The diagrams covering rigid building frames involved an enormous amount of computation, due in part to the scarcity of previously published data on the subject, in part to the complexity involved in the consideration of both columns and girders, and in part to the variation in the size and stiffness of the columns as compared with the stiffness of the girders framing into them.

The following structural conditions are involved in the analyses for the rigid building frame diagrams:

(1) Intermediate story, with the wall-end of the girder framing into a wall column (columns both above and below the girder).

(2) Top story, with the wall-end of the girder framing into a wall column (columns below the girder only). This is a special case of that in paragraph (1) as explained in the notes on the diagrams.

(3) Intermediate story, with the wall-end of the girder freely supported (columns both above and below the girder).

(4) Top story, with the wall-end of the girder freely supported (columns

below the girder only). This is a special case of that in paragraph (3), as explained in the notes on the diagrams.

It was considered unnecessary to analyze these frames beyond the third span from the end, on the assumption that for all practical purposes additional interior spans might be designed on the basis of the moment and shear values in the third span. The later analyses indicated that this assumption is correct.

For a structure having only three spans, a close approximation will be obtained by using the curves in the first span and the left half of the second span, then using the same curves for the other half of the structure, considered reversed.

If a structure has four spans, design the first two spans in accordance with the first two spans of the diagrams; then consider the structure reversed and design the remaining two spans in the same way, namely, in accordance with the first two spans of the diagrams.

The diagrams are not directly applicable to the design of a structure of two spans, although a fairly close approximation will be obtained by designing each span in turn in accordance with the first span of the diagrams.

The rigid frame analyses are based on the work of S. C. Hollister, published in Section 10 of Hool and Johnson's "Concrete Engineers' Handbook." Mr. Hollister's work is based on the slope-deflection method of analysis developed with the aid of the principle of area moments. In so far as the final result is concerned, his work deals only with spans of equal length; or, more properly stated, with the case where the stiffness factors of the girder spans are equal. It was therefore necessary to amplify his analysis to the extent of developing more general formulas applying to unequal spans and to columns of unequal size.

In order to obtain the results desired, formulas were developed for the general case of unequal span lengths (or girders having unequal stiffness factors), with columns above and below the girders having unequal stiffness factors, and with adjoining columns in the same story having unequal stiffness factors. From these general formulas, which have been omitted from this paper, the actual moment and shear values for the special case of girders with equal stiffness factors (practically equal spans) and columns in the same story having equal stiffness factors were

then computed. The curves were plotted from those values for equal spans, as shown in the diagrams.

Further modifications were made in the bending moment curves in half of the rigid frame diagrams as the result of a careful check by G. C. Staehle, structural engineer in the general office of the Portland Cement Association. He employed independent computations of a more complex character. His computations agreed very closely with the author's computations for the rigid frame set of diagrams in which the wall-end of the girder is freely supported.

For the diagrams covering the case where the wall-end of the girder frames into a wall column, Mr. Staehle's computations indicated a somewhat greater difference in moment values, the amount of difference being appreciably large in several cases.

In general, Mr. Staehle's computations indicated an increase in the bending moments at the first interior support, a decrease in the bending moments at the wall-end and at the right end of the second span, and a negligible decrease at both ends of the third and remaining interior spans.

Expressed graphically, the bending moment curves were tilted to the left in the first span and to the right in the second span.

The greatest differences in value occurred at the first interior support in Diagrams 5 to 11, inclusive, where the negative bending moment was appreciably larger in Mr. Staehle's computations. In view of the higher degree of accuracy in the method used by him, the curves were modified to conform to his results.

EXTENT OF RIGID FRAME ANALYZED

The extent of the rigid frame considered in the analysis and computations for the moment and shear diagrams is shown in Fig. 1. The frame analyzed consists of two pairs of columns and

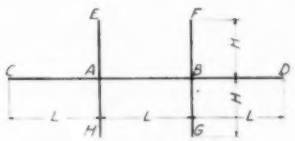


FIG. 1—EXTENT OF RIGID FRAME CONSIDERED IN ANALYSIS.

a line of three successive girder spans.

On first impression the designer may feel that the extent of this frame is too restricted. For instance, if we were to consider the structural frame of a building such as the one shown diagramatically in Fig. 2, we would get the maximum positive bending moment at the middle of the girder span AB by placing a live load as shown, on alternate panels throughout all stories of the building. This maximum bending moment would occur on the further assumption that the remaining alternate panels carried no live load.

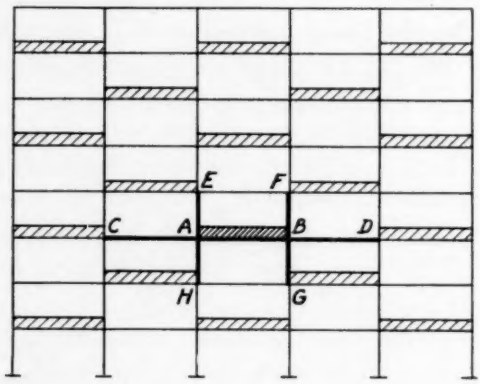


FIG. 2—DISTRIBUTION OF LIVE LOAD FOR THEORETICALLY MAXIMUM POSITIVE BENDING MOMENT IN GIRDER A-B.

There are two reasons why the calculations should not be carried to so great an extreme. First, it would require an unusual stretch of the imagination to draw a mental picture of a building with so perfect a load distribution. In the second place, the loads in the remoter parts of the structure would have an infinitely small effect on girder AB.

The truth of the latter statement may be realized when we consider what effect a uniformly distributed load on girder AB will have on a span so near at hand as the adjoining span AC.

Computations disclosed that with the sum of the stiffness factors of girders *AB* and *AC* equal to the sum of the stiffness factors of Columns *AE* and *AH*, the bending moment at the left end of girder *AB* is $7/96wL^2$, and the sum of the bending moments in the two columns is $6/96wL^2$, leaving an unbalanced bending moment of only $1/96wL^2$ to be taken by the right end of girder *AC*.

The relation between the sum of the stiffness factors of the columns and the sum of the stiffness factors of the girders used in the foregoing illustration is the relation likely to occur at about the second or third story from the top of a building. In stories further down, where the sum of the stiffness factors of the columns may be five or ten times that of the girders, practically all of the bending moment at the left end of girder *AB* (due to a uniform load on that girder) will be resisted by the columns, very little being carried across to girder *AC*. For instance, if the sum of the stiffness factors of columns AE and AH is five times the sum of the stiffness factors of girders *AB* and *AC*, a uniformly distributed load on girder *AB* will produce a bending moment of $31/384wL^2$ at the left end of girder *AB*, $30/384wL^2$ in the two columns, and only $1/384wL^2$ at the right end of girder *AC*.

From the two foregoing illustrations the author felt justified in agreeing with Mr. Hollister's conclusions that the frame shown in Fig. 1 is a unit of sufficient extent for all practical purposes.

RANGE IN RATIO BETWEEN COLUMN STIFFNESS AND GIRDER STIFFNESS

The next step comprised a consideration of the probable extremes in the ratio between the stiffness factors of columns and the stiffness factors of the girders framing into them. In the general formulas the terms representing the influence of the columns above and below the girder always appeared in pairs and with prefixes of equal value. Without going into further discussion of this point, it will be sufficient to refer to the effect of the fact just stated. This will be found in Note No. 6 on the diagrams. As stated in that note, the column moments shown in the diagrams are based on the assumption of equal stiffness in the columns above and below the girder. The lower of two columns in the same vertical line, if they are of the same free length,

naturally has the greater stiffness factor; but the sum of the bending moments taken by the two columns remains unchanged so long as the sum of their stiffness factors remains unchanged.

The foregoing statement can be made more understandable by an actual illustration. Refer, for example, to Diagram 8, and consider the upper and lower columns at the second interior support. To fit the case of this particular diagram, assume that the stiffness factor of each girder is 45, that of the upper column is 36, and that of the lower column is 54. Thus the sum of the stiffness factors of the columns is equal to that of the girders. Assume further that the girders have a clear span of 20 ft., that the dead floor load is 1,000 lb. per lin. ft., and that the live floor load is twice as much, or 2,000 lb. per lin. ft. On the basis of these assumptions the diagram shows a maximum bending moment of $0.02\ wL^2$, or 24,000 ft.-lb., in each column, making a total of 48,000 ft.-lb. in the two columns. As perviously stated, the actual bending moment taken by either column is in direct proportion to its stiffness factor. Consequently the upper column will take 36/90 of the total, or 19,200 ft.-lb., while the lower column will take 54/90 of the total, or 28,800 ft.-lb.

Shears in columns were not plotted, inasmuch as their values may be obtained by dividing the bending moment in any column by the distance from the section of maximum moment in the column to its point of contra-flexure.

Returning now to the probable range of the ratio between the stiffness of columns and the stiffness of girders, the author studied the plans for a reinforced concrete building 16 stories high, designed for a live load of 150 lb. per sq. ft., and with floor girder spans of about twenty feet. The moment of inertia of a typical roof girder and a typical floor girder were computed and the stiffness factor of each was obtained by dividing the moment of inertia (expressed in inches to the fourth power) by the free length (expressed in inches). Similar computations were made with respect to a typical vertical line of columns.

From these computations it was learned that the stiffness factor of the top-story column was only about 1/5 of the stiffness factor of the roof girder. In the third story from the top the stiffness factor of the column was slightly greater than that of the typical floor girders. In the basement story the stiffness

factor of the column was about fifteen times the stiffness factor of the floor girders.

Trial computations disclosed that even in cases where the stiffness factor of the column is five times that of the girder the condition approaches very closely to the condition of complete rigidity. It was therefore not considered necessary to carry the comparison beyond a ratio of ten. That is to say, when the sum of the stiffness factors of the columns is 10 times the sum of the stiffness factors of the girders framing into them, the moments and shears in those girders are for all practical purposes the same as if the ends of the girders were completely fixed.

In view of the foregoing, the values of bending moments and shears were computed for seven different relations between column stiffness and girder stiffness. These seven relations, as shown on the diagrams, include the following:

(1) The sum of the stiffness factors of the columns is *one-tenth* the sum of the stiffness factors of the girders.

(2) The sum of the stiffness factors of the columns is *one-fifth* that of the girders.

(3) The sum of the stiffness factors of the columns is *one-half* that of the girders.

(4) The sum of the stiffness factors of the columns is *equal* to that of the girders.

(5) The sum of the stiffness factors of the columns is *two and one-half times* that of the girders.

(6) The sum of the stiffness factors of the columns is *five times* that of the girders.

(7) The sum of the stiffness factors of the columns is *ten times* that of the girders.

These seven different relations between column stiffness and girder stiffness, combined with the four different structural conditions previously mentioned, required the construction of 14 diagrams to cover all cases. The frame resulting from the roof girders and the top story columns constitutes a special condition readily solved from the diagrams.

NOTES ON DIAGRAMS

Before discussing the actual use of the diagrams it might be well to discuss some of the notes on the diagrams and the nature of the loading considered. For instance, the symbol w is defined in the notes as the uniformly distributed dead load and live load combined, in pounds per linear foot, with the live load placed in

the most unfavorable position. In the actual distribution of the live load, a span was considered as either fully loaded or fully unloaded, except that for the determination of the maximum and minimum shears at mid-span the span under consideration was loaded only on the right or left half.

The preceding paragraph may well be amplified. For example, in a system of continuous beams of five spans the maximum positive bending moment in the first span (not far from the four-tenths point, measured from the left end) is obtained by considering the first, third, and fifth spans loaded, and with the second and fourth spans unloaded. The dead load, of course, appears on all spans. The maximum negative bending moment at the first interior support is obtained by placing live loads on the first, second and fourth spans, and considering the third and fifth spans unloaded. The maximum positive shear at the left end of the first span is obtained by considering the first, third and fifth spans loaded and the second and fourth spans unloaded. The maximum negative shear at the right end of the first span is obtained by having the first, second and fourth spans loaded and the third and fifth spans unloaded. At the middle of the first span the maximum positive shear is obtained by placing live loads on the third and fifth spans and on the right half of the first span, with other parts unloaded. The maximum negative shear at mid-span is obtained by placing the live load on the second and fourth spans and on the left half of the first span, with other parts unloaded.

All the diagrams contain a foot-note to the effect that L is the span length in feet. This has reference, of course, to the computations for bending moments and shears. In computing the stiffness factor of girders and columns the moment of inertia will presumably be figured in inches to the fourth power, in which event the free length of the girder or the column under consideration must be expressed in inches.

Another note that may call for further explanation is number (5) on the rigid frame diagrams, in which formulas are given for computing the moment of inertia of reinforced concrete columns and girders. The formulas there given are those adopted by the New York Sub-Committee on Reinforced Concrete. Designers

may have differences of opinion as to the correctness of the formulas given. The important point to remember is that the formulas for moment of inertia as here given do not in any way affect the correctness of the moment or shear values in the diagrams. The formulas for moment of inertia are given only as a matter of information and convenience.

The only remaining notes on the diagrams that require special explanation are those defining the significance of the numbers *0*, *1*, *2* and *3* (in small circles) pointing to the bending moment and shear curves.

The curves marked *0* (in a small circle) indicate the moment and shear values obtained either from the dead load or from the dead load combined with a live load uniformly distributed on all spans. The value of w is, of course, the dead load alone in the former case and the combined dead load and live load in the latter case.

The curves marked *1* (in a small circle) represent the maximum and minimum bending moments and shears in the case where the live load is equal to the dead load, both expressed in lb. per lin. ft. In this case w is the combined dead load and live load, in lb. per lin. ft.

Curves marked number *2* (in a small circle) represent the maximum and minimum bending moments and shears obtained when the live load is twice the dead load. As before, w is the combined dead load and live load expressed in lb. per lin. ft.

The curves marked number *3* (in a small circle) represent the maximum and minimum bending moments and shears obtained when the live load is three times the dead load. Here, as before, w is the combined dead load and live load, in lb. per lin. ft.

For any other ratio between live load and dead load, the designer should interpolate between two of the curves. For illustration, when the live load is half the dead load, the values will be obtained by interpolating between curve number *0* and curve number *1*.

POINTS OF INFLECTION

One of the most valuable features of these diagrams lies in the fact that the various positions of the point of inflection may be located visually. Take, for illustration, the diagram for a con-

tinuous beam of three equal spans and assume that the live load is equal to the dead load. Number *1* curve applies in this case. From Diagram 2 it will be seen that with the live load distributed so as to produce the maximum positive bending moment in the first span, the point of inflection is about 0.15L from the right end of that span; but with the live load placed so as to produce the least bending moment near the middle of the first span, the point of inflection will be 0.3L from the right end of that span.

Considering the middle span on the same diagram, the designer will see at once that the point of inflection, when the system is loaded for maximum positive moment in that span, will be at about 0.18L from the right and left ends; but when the system is loaded so as to produce the maximum negative moment at the middle of the span there is no point of inflection, the entire span being subjected to negative bending. This negative bending over the full length of the middle span occurs when the two end spans are loaded and the middle span is unloaded.

NEGATIVE MOMENT OVER ENTIRE SPAN

This question of having negative bending over an entire span is not a matter of theory. Members of the New York Sub-Committee cited instances in which floor girders and slabs of heavily loaded reinforced concrete buildings showed ample evidence that negative bending does occur over an entire span under certain conditions of loading. Consider, for example, a warehouse loaded heavily with merchandise. Shipments are made, with the result that one floor bay is completely emptied while the two adjoining bays remain heavily loaded. Under such conditions the unloaded panel may be under quite heavy negative bending over the entire span. If the designer has turned down the top steel too soon, or if he has not placed some top steel over the entire span, cracking in the girders and in the adjacent floor slabs is almost sure to occur.

USE OF DIAGRAMS

The use of these diagrams in determining the maximum and minimum bending moments and shears and points of inflection will now be illustrated by actual examples, round members being assumed for convenience.

ILLUSTRATIVE EXAMPLE, CONTINUOUS BEAM DIAGRAMS

The first example will be that of a continuous beam of three equal spans (Diagram 2) with an assumed live load equal to twice the dead load. For convenience, assume a dead load of 1,000 lb. per lin. ft., a live load of 2,000 lb. per lin. ft., and a span length of 20 ft. from *face to face* of supports. The supports in this case will be the supporting wall at the end, with intermediate supports consisting of the girders into which the continuous beams are framed. Number *2* curves apply to the condition assumed, namely, that the live load is twice the dead load.

Consider the first span. The bending moment shown at the left end is the arbitrary bending moment of $-1/24wL^2$ (or $-0.0417wL^2$) selected for the purpose of reinforcing the beam against partial fixity at the wall-end. This arbitrary bending moment is disregarded in the analysis of the remainder of the system.

The maximum positive bending moment in the first span occurs at about 0.42L from the left end, with a value of about $0.094wL^2$.

The maximum negative bending moment at the right end of the first span is about $-0.111wL^2$. The maximum negative bending moment at the left end of the second span has, of course, this same value of $-0.111wL^2$.

The maximum positive bending moment at the center of the second span has a value of $0.058wL^2$, and the maximum negative moment at the same point is $-0.025wL^2$. Negative bending may occur over the entire span.

The *point of inflection* near the left end of the first span is purely arbitrary, inasmuch as it is determined from the arbitrary bending moment value of $-1/24wL^2$, assumed for the purpose of taking care of partial fixity at the freely supported end.

Near the right end of the first span, the point of inflection is 0.14L from the right end of the span when the live load is placed on the first and third spans only; but when the live load is placed on the middle span only, and the first and third spans are unloaded, the point of inflection is 0.4L from the right end of the span.

In the second span, the point of inflection may occur as near

as 0.15L from the face of the support when that span is loaded and the two outer spans are unloaded.

When the two outer spans are loaded and the middle span is unloaded, there is no point of inflection in the second span. The entire span is subjected to negative bending.

SHEAR VALUES

Referring now to the shear curves, it will be seen that, for the loading assumed, the maximum shear at the left end of the first span is 0.433wL, and —0.611wL at the right end.

At the mid-point of the first span there may be a maximum positive shear of 0.036wL or a maximum negative shear of about —0.170wL.

In the second span the maximum positive shear at the left end is 0.555wL, while the maximum negative shear at the right end has the same numerical value but is of opposite sign.

At the mid-point of the second span there may be either a positive or a negative shear of as much as 0.132wL.

CHARACTER OF SHEAR CURVES

In connection with the shear curves, the manner in which they are obtained requires further explanation. Consider, for illustration, the curve indicating the positive shear in the first span on Diagram 2 (three equal spans). First, the maximum positive shear at the left end was obtained in the usual way by assuming a live load on the first and third spans and considering the middle span unloaded. The shear value thus obtained was plotted.

The maximum positive shear at the middle of the first span was then computed by assuming the live load placed on the third span and on the right half of the first span, and assuming the middle span unloaded. This shear value was also plotted.

The two points thus plotted (representing the maximum positive shear at the left end and at the middle) were joined by a straight line which was continued to the base line representing the beam.

If the shear had been computed at each of the tenth points between the left end and the middle of the span, values varying slightly from the straight line would have been obtained; but the variation would be too small to be worth considering. Such

maximum shear values at any point between the left end and the middle of the first span would have been obtained by placing a live load on the entire third span and on that part of the first span extending from the point under consideration to the first interior support, and with the middle span unloaded. Such a perfect condition of loading is not likely to occur, so that the straight shear line, as located, is sufficiently close for all practical purposes.

ILLUSTRATIVE EXAMPLE—RIGID FRAME DIAGRAMS

The next illustrative example will deal with the use of the fourteen diagrams for obtaining bending moments, shears and points of inflection in rigid building frames.

For this example consider the same loading conditions, namely, a dead load of 1,000 lb. per lin. ft. and a live load of 2,000 lb. per lin. ft., the live load again being distributed so as to produce the maximum positive or negative bending moments or shears at the various points under consideration.

For the purpose of this illustration take Diagram 15, in which the sum of the stiffness factors of the columns is equal to the sum of the stiffness factors of the girders. This is the relation likely to occur at a point two or three stories from the top of the building. The wall-end of the girder is freely supported, but is reinforced for an arbitrary bending moment of $-1/24wL^2$, to take account of partial fixity. As in the case of continuous beams, this arbitrary end moment does not enter into the remaining computations. Number *2* curves cover the condition assumed, namely, that the live load is twice the dead load.

The maximum positive bending moment in the first span occurs at a point nearly 0.4L from the left end, and it has a value of about $0.0775wL^2$. The maximum negative bending moment at the right end of the span is about $-0.1175wL^2$.

The maximum negative bending moment at the left end of the second span is not the same as the maximum negative moment at the right end of the first span. The difference in these bending moment values must be taken up by the columns.

In the example under consideration, the maximum negative bending moment is about $-0.095wL^2$ at the left end of the second span, and about $-0.079wL^2$ at the right end.

Near the middle of the second span the maximum positive bending moment is about $0.048wL^2$. The negative moment does not extend quite over the entire span, so that even for the most unfavorable condition of loading there is a small amount of positive moment at the center of the second span.

In the third span the maximum negative moment at the left and right ends is about $-0.0815wL^2$.

The maximum positive moment at the middle of the third span is about $0.050wL^2$, and again there is no negative moment at this point.

POINTS OF INFLECTION

In the first span, still considering the case where the live load is twice the dead load, the point of inflection may approach within 0.22L of the right end. Under a more unfavorable condition of loading, it may be 0.29L from the right end. This latter value is shown by the short line marked number *2* (in a small circle) which represents a small sector of the bending moment curve producing the minimum positive moment near the middle of the first span. This curve represents the results obtained by having the second span loaded and the first and third spans unloaded.

In the second span the point of inflection ranges from about 0.19L to 0.40L from the left end. Measured from the right end, it ranges from 0.18L to 0.38L.

BENDING MOMENTS IN COLUMNS

The bending moments in columns are represented by short straight lines drawn up or down from the base line representing the girders. They are drawn to the same scale as the bending moment curves for the girders, and are expressed as coefficients of wL^2, w being the unit beam load.

As pointed out in Note (6), and as explained in another part of this text, the column bending moments shown on the diagram, when added together, give a true value of the sum of the actual bending moments taken by the two columns. However, the stiffer of the two columns will take the greater part of the total moment.

Referring now to Diagram 15, the *sum* of the maximum bending

moments in the upper and lower columns at the first interior support is $0.068wL^2$. Assuming that the stiffness factor of the upper column is 36 and that of the lower column is 54, the upper column will take 36/90 of the total bending moment, while the lower will take 54/90 of the total.

At the second interior support the *sum* of the maximum bending moments in the upper and lower columns is $0.045wL^2$. The portion taken by each column will be in proportion to its stiffness factor.

At the third interior support the *sum* of the maximum bending moments in the upper and lower columns is $0.042wL^2$. As before, each column will take a part of the total in proportion to its stiffness factor.

SHEAR VALUES

The shear values in the girders are obtained in the same manner as with continuous beams. The shear values in the columns are not plotted, since they may be obtained by dividing the bending moment in a column by the distance from the section of maximum moment in the column to its point of contra-flexure.

CUT-AND-TRY METHOD NOT NECESSARY

For the reason that the bending moments, shears and points of inflection in a rigid frame are dependent in part on the relation between the stiffness of the columns and the stiffness of the girders, one is likely to get the impression that cut-and-try methods are necessary in order to determine in advance what this relation will be in any given case. On the contrary, such a procedure is not required.

Fig. 3 is a design aid, or work sheet, from which the designer may obtain at once the bending moments in the columns, and at the ends and mid-span of the girders, for any ratio between column stiffness and girder stiffness. Fig. 3 applies specifically to cases where the wall-end of the girder frames into a wall colmun; that is, to Diagrams 5 to 11, inclusive.

The designer should draw up a similar work sheet covering shears and points of inflection. Likewise, additional work sheets will be required for bending moments, shears and points of

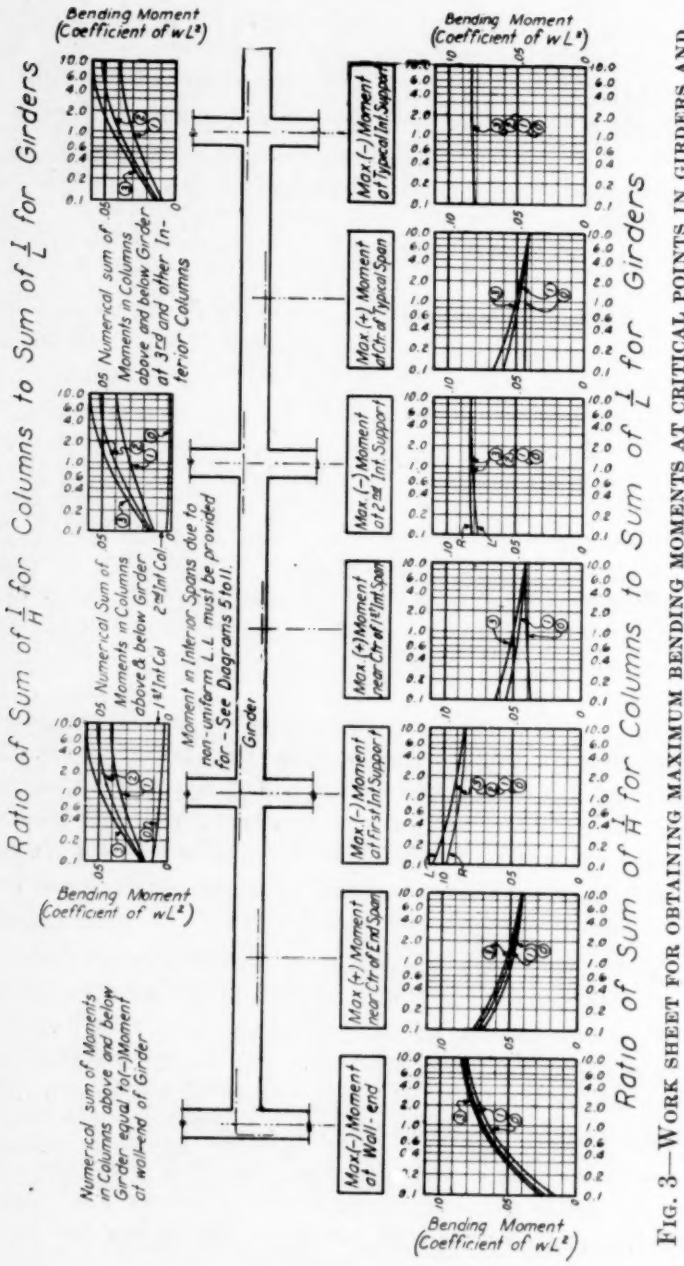

Ratio of Sum of $\frac{I}{H}$ for Columns to Sum of $\frac{I}{L}$ for Girders
Bending Moment (Coefficient of wL^2)
Numerical sum of Moments in Columns above and below Girder at 3rd and other Interior Columns
Numerical Sum of Moments in Columns above & below Girder
1st Int Col
2nd Int Col
Moment in Interior Spans due to non-uniform L.L. must be provided for – See Diagrams 5 to 11.
Girder
Numerical sum of Moments in Columns above and below Girder equal to (–) Moment at wall-end of Girder
Max (–) Moment at Wall-end
Max (+) Moment near Ctr of End Span
Max (–) Moment at First Int Support
Max (+) Moment near Ctr of 1st Int Span
Max (–) Moment at 2nd Int Support
Max (+) Moment at Ctr of Typical Span
Max (–) Moment at Typical Int Support
Ratio of Sum of $\frac{I}{H}$ for Columns to Sum of $\frac{I}{L}$ for Girders
Bending Moment (Coefficient of wL^2)

FIG. 3—WORK SHEET FOR OBTAINING MAXIMUM BENDING MOMENTS AT CRITICAL POINTS IN GIRDERS AND COLUMNS (DIAGRAMS 5 TO 11 INCL.).

inflection in a design problem covered by Diagrams 12 to 18, inclusive.

The construction and use of Fig. 3 will now be explained. Consider, for instance, the first small rectangular diagram in Fig. 3, giving the bending moment coefficients at the wall-end of the girder in the first span. As in Diagrams 1 to 18, inclusive, the curves marked *0*, *1* and *3* (in small circles) indicate moment coefficients for dead load alone, for live load equal to the dead load, and for live load three times the dead load. Curve *2* is omitted because it lies so close to curve *3* that no distinction could be made on so small a scale.

The curves in the small rectangular diagram are now plotted by taking the moment coefficient values at the wall-end of the girder in the first span directly from Diagrams 5 to 11, inclusive, thereby locating seven points on each curve. With the curves drawn, the designer need no longer refer to the principal diagrams (Diagrams 5 to 11, inclusive). The curves in the small rectangular diagrams in Fig. 3 will give him the bending moments at all critical points in the girders and columns, for any ratio between column stiffness and girder stiffness ranging from 0.1 to 10.

All other curves in Fig. 3 are, of course, plotted in the manner described above in connection with the first small diagram. In the column moment diagrams the curves are plotted also for the case where the live load is twice the dead load, since the distance between curves *2* and *3* is sufficient to make a distinction between them.

Attention is invited to the two curves marked *L* and *R* of the three small diagrams in Fig. 3 which give the maximum bending moment coefficients at interior supports. The curves marked *L* represent the maximum bending moment coefficients at the girder-end at the left face of the column. The curves marked *R* give the maximum bending moment coefficients at the girder-end at the right face of the column.

It is well to mention the fact that Fig. 3 does not include curves representing minimum bending moments at the mid-span of girders. These minimum mid-span moments may in some cases

be negative, as the Diagrams show. The designer might well include curves covering this case when preparing work sheets similar to Fig. 3.

CONTINUOUS BEAMS OF UNEQUAL SPAN

When the longer of any two adjoining spans of a system of continuous beams does not exceed the shorter span length by more than 20 per cent, the bending moments, shears and points of inflection may be obtained by the use of the continuous beam diagram for the same number of equal spans, by the following method:

1. For both left and right maximum end shears in end spans assume the system of beams to be made up of all equal spans of the actual length of the end span in question. For example, consider Fig. 4, a system of continuous beams of three unequal spans with an assumed live load equal to twice the dead load. Determine the left or right end shear in the 16-ft. span by taking the shear coefficient at the corresponding point directly from Diagram 2, using the shear curve marked number *2* (in small circle). Use the actual length of the given span as the value of L in the shear coefficient thus obtained. The left or right end shear in the 24-ft. span is obtained in the same manner by using the actual length of the span in question for L.

2. For maximum end shears in interior spans assume the system of beams to be made up of all equal spans of the average length of the span under consideration and the span on the opposite side of the support at which the shear is desired. For example, consider Fig. 4. To determine the shear at the left end of the 20-ft. span take the shear coefficient at the corresponding point directly from Diagram 2, using the shear curve marked number *2*, (in small circle). Use the average length of the 20-ft. span and the 16-ft. span as the value of L in the shear coefficient thus obtained. The right end shear in the 20-ft. span is obtained in the same manner by using the average length of the 20-ft. span and the 24-ft. span as the value of L.

3. For maximum and minimum shears at mid-span assume the system of beams to be made up of all equal spans of the actual length of the span in question. For example, consider Fig. 4. To determine the shear at the center of any one of the spans take the shear coefficient at the corresponding point directly from Diagram 2, using the shear curve marked *2* (in small circle). Use the actual length of the given span as the value of L in the shear coefficient thus obtained.

4. For bending moments over supports assume the system of beams to be made up of all equal spans of the average length of the two spans adjoining the support. For example, consider Fig. 5. To determine the maximum moment at the first interior support (left) take the moment coefficient at the corresponding point directly from Diagram 2, using the negative moment curve marked

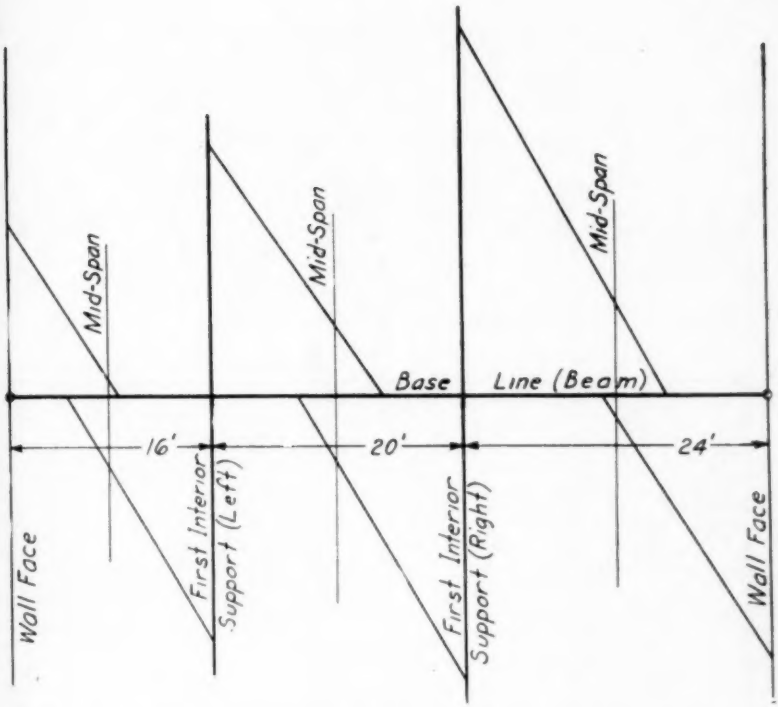


FIG. 4—ILLUSTRATING METHOD OF OBTAINING APPROXIMATE SHEAR VALUES IN 3 UNEQUAL SPANS.

2 (in small circle). Use the average length of the 16-ft. span and the 20-ft. span as the value of L in the moment coefficient thus obtained. The maximum negative moment at the next support is obtained in the same manner by using the average length of the 20-ft. span and the 24-ft. span as the value of L in the moment coefficient.

5. For both maximum and minimum bending moments at or near mid-span, use the followng method:

(a) Lay off the actual span lengths to a convenient scale, as in Fig. 5. Obtain the moments at the supports by the method given in paragraph 4 above and plot these values to scale. (These values are to be the actual moments, not moment coefficients). Draw straight lines between the points just plotted, as shown in Fig. 5.

(b) Draw a similar straight line between the corresponding points on the moment curves on Diagram 2, for this example using the curves marked *2* (in small circle).

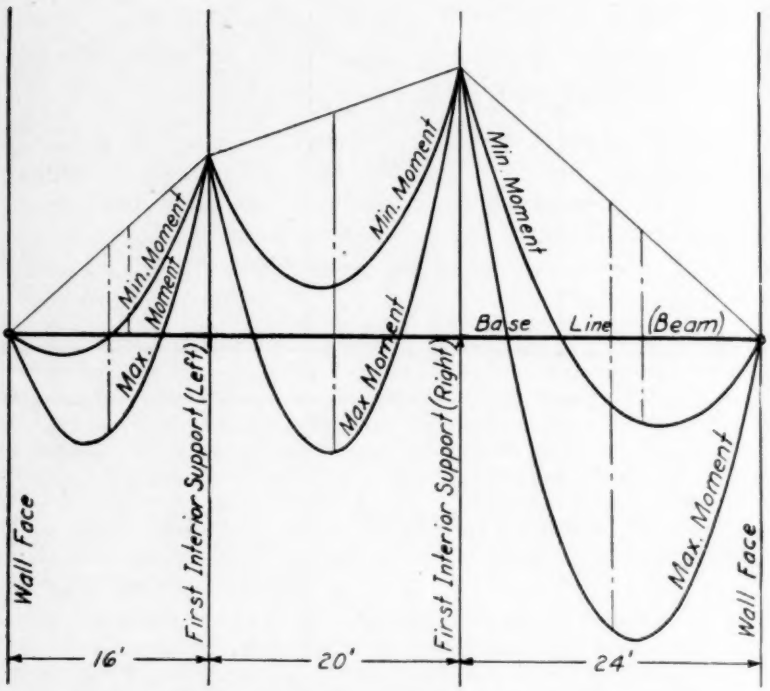
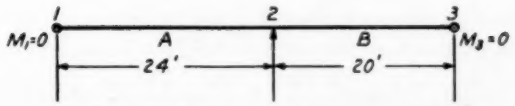


FIG. 5—METHOD OF OBTAINING APPROXIMATE BENDING MOMENTS IN 3 UNEQUAL SPANS.

(c) For any span in Fig. 5 obtain the bending moment *coefficient* for any convenient point in that span by measuring the vertical ordinate (at the corresponding point on Diagram 2) between the straight line just drawn on Diagram 2 and the maximum and minimum bending moment curves.

(d) From the bending moment *coefficients* just obtained from Diagram 2 compute the *actual* bending moments at the points selected in Fig. 5, using the actual length of the span for L, and plot these moments to scale at the corresponding points on the layout of unequal spans (Fig. 5). Connect with parabolic curves the points established for maximum moment at the supports and the points just located. The ordinates at mid-span measured between the girder line and the curves just drawn give the maximum and minimum bending moments at that point.



CONTINUOUS BEAM – 2 UNEQUAL SPANS

Live Load = 2 x Dead Load Assume L.L. = 2,000 lb. per lin. ft.
D.L. = 1,000 lb. per lin. ft. Total w = 3,000 lb. per lin. ft.

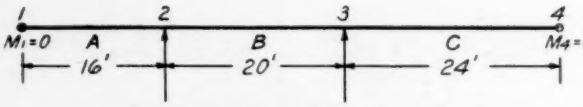
	DESIGNATION	DEAD LOAD ONLY		DEAD LOAD & LIVE LOAD COMBINED			
		ACTUAL VALUE	VALUE BY APPROX. METHOD	ACTUAL VALUE		VALUE BY APPROX. METHOD	
				MAX.	MIN.	MAX.	MIN.
		(1)	(2)	(3)	(4)	(5)	(6)
Moment at Middle of First Span.	M_A	+41,000	+41,750	+145,730	+18,270	+145,840	0
Moment over Middle Support.	M_2	-62,000	-60,500	-186,000	—	-181,500	—
Moment at Middle of Second Span.	M_B	+19,000	+19,750	+96,270	-20,270	+101,280	0
Shear at Left End	V_1	+ 9,420	+ 9,000	+28,255	—	+30,000	—
Shear at Left of Middle Support	V_{2L}	- 14,580	- 15,000	-43,745	—	-45,000	—
Shear at Right of Middle Support	V_{2R}	+13,100	+12,500	+39,300	—	+45,000	—
Shear at Right End	V_3	- 6,900	- 7,500	-20,700	—	-25,000	—

FIG. 6—MOMENTS AND SHEARS IN TWO UNEQUAL SPANS.

Figs. 6 and 7 give the true bending moments and shears in the cases of two unequal spans and three unequal spans, and the corresponding values obtained by utilizing the diagrams in accordance with the approximate method just described. In the two-span system the longer span is 20 per cent in excess of the shorter. In the three-span system the span length of the longer span exceeds the shorter by 25 per cent in one case and 20 per cent in the other.

RIGID FRAMES WITH GIRDERS OF UNEQUAL SPAN

The diagrams for rigid frames, like those applying to continuous beams, may be utilized in cases where the girder spans are unequal.



CONTINUOUS BEAM – 3 UNEQUAL SPANS

Live Load = 2 x Dead Load Assume L.L. = 2,000 lb. per lin ft.
D.L. = 1,000 lb. per lin. ft. Total w = 3,000 lb. per lin. ft.

	DESIGNATION	DEAD LOAD ONLY		DEAD LOAD & LIVE LOAD COMBINED			
		ACTUAL VALUE	VALUE BY APPROX. METHOD	ACTUAL VALUE		VALUE BY APPROX. METHOD	
				MAX.	MIN.	MAX.	MIN.
		(1)	(2)	(3)	(4)	(5)	(6)
Moment at Middle of First Span.	M_A	+18,775	+15,800	+79,240	- 4,135	+70,400	+ 6,400
Moment over Second Support	M_2	-26,450	-32,400	-102,630	—	-108,000	—
Moment at Middle of Second Span.	M_B	+ 8,775	+10,000	+ 68,345	-33,235	+ 70,000	-30,000
Moment over Third Support.	M_3	-55,990	-48,400	-174,870	—	-161,330	—
Moment at Middle of Third Span	M_C	+44,000	+47,800	+149,525	+26,480	+158,400	+ 7,200
Shear at Left End	V_1	+ 6,350	+ 6,400	+21,905	—	+ 20,800	—
Shear at Left of Second Support	V_{2L}	- 9,650	- 9,600	-30,410	—	- 29,335	—
Shear at Right of Second Support	V_{2R}	+ 8,525	+ 9,000*	+30,925	—	+ 30,000*	—
Shear at Left of Third Support	V_{3L}	-11,475	-11,000*	-36,295	—	36,665*	—
Shear at Right of Third Support	V_{3R}	+14,335	+14,400	+43,285	—	+ 44,000	—
Shear at Right End	V_4	- 9,665	- 9,600	-30,460	—	- 31,200	—

*Average of 2 spans adjoining support

FIG. 7—MOMENTS AND SHEARS IN THREE UNEQUAL SPANS.

If the sum of the stiffness factors of the upper and lower columns is at least five times that of the girders, any individual span may be considered as rigidly fixed at the column ends, and the bending moments and shears may be obtained from the corresponding moment and shear diagrams for equal spans.

If the sum of the stiffness factors of the upper and lower columns is at least equal to that of the girders, the longer of any two adjoining spans may be as much as 50 per cent in excess of the shorter span. In this case, as before, bending moments and shears may be obtained from the corresponding moment and shear diagrams for equal spans.

If the sum of the stiffness factors of the upper and lower columns is one-tenth of the sum of the stiffness factors of the girders, the diagrams for equal spans may be used if the longer of any two adjoining spans does not exceed the shorter by more than 25 per cent.

The limits of span variation just established may be considered as guides for the allowable variation with other relations between column stiffness and girder stiffness.

In all of the foregoing cases applying to rigid frames, the assumed equal spans shall be taken as the actual span length of the girder under consideration.

Unequal girder moments on opposite sides of a support, to the extent that such unequal moments are due to the difference in actual span length of the two adjoining girders, must be added to the normal bending moments in the columns. The normal bending moments in the columns are those obtained from the diagrams, which are, of course, based on equal girder spans.

The foregoing paragraph can perhaps be understood better through reference to one of the diagrams. Refer, for illustration, to the first interior support in Diagram 6. Note that the maximum bending moment at the right end of the girder in the first span exceeds, by about $0.007wL^2$, the maximum bending moment at the left end of the girder in the second span. This unbalanced moment of $0.007wL^2$ is taken up by the columns and is the normal bending moment in the columns. Suppose, now, that the first span is longer than the second, so that the maximum bending moment at the right end of the first span will be increased considerably. The moment at the left end of the second span will also be increased, but to a smaller extent, so that the difference between the two girder-end moments will be considerably greater than $0.007wL^2$. Assume that the first span length is such that this unbalanced moment is $0.017wL^2$ (instead of $0.007wL^2$).

This increase of $0.010wL^2$ in the unbalanced girder-end moments must be added to all the column moments at this support, not only to the one representing the dead load, but also to the moment that includes the effect of the live load.

COLUMNS IN SAME STORY HAVING UNEQUAL STIFFNESS

When the sum of the stiffness factors of the columns at one end of a girder span differs from that of the columns at the other end, the ratio between column stiffness and girder stiffness at one end of the span will of course be different from that at the other end. In such a case the bending moments and shears at the left end of the girder and in the columns at the left end should be based on the stiffness ratio between columns and girders at that end. Likewise, the bending moments and shears at the right end of the girder and in the columns at the right end should be based on the stiffness ratio between columns and girders at that end. For moments and shears at mid-span, assume a stiffness ratio between columns and girders equal to the average of the ratios existing at the left and right ends.

ACKNOWLEDGMENT

Grateful acknowledgment is made of valuable aid received from R. L. Bertin, Chief Engineer of the White Construction Co., New York; from M. J. Roach, Engineer for the New York Office of the Turner Construction Co.; from Arthur R. Lord, Consulting Engineer, Chicago; and from G. C. Staehle and L. H. Corning, structural engineers in the general office of the Portland Cement Association, Chicago.

Mr. Corning checked the author's computations, while Mr. Staehle made the independent computations previously described.

Mr. Bertin and Mr. Roach, both of whom were members of the New York Sub-Committee on Reinforced Concrete, and Mr. Lord were in large part responsible for the manner of arrangement of the moment and shear curves, and they checked some of the bending moment values against values obtained by other methods of approach. In addition, they all had the diagrams tried out by designers in their offices, from whom further suggestions on arrangement were received.

RANGE OF USEFULNESS

While these diagrams are devised with special reference to

uniformly distributed loads, their range of usefulness may be extended to include other types of loading which can be converted into approximately equivalent uniform loading. For illustration, bending moments due to load concentrations at the third-points may be closely approximated by such conversion, but the shear values for this case must be obtained by other methods. Diagrams to cover the special case of load concentrations at the third points would constitute a helpful addition to the diagrams here presented.

Another fact well worth remembering is that the use of these diagrams is not limited to structures of reinforced concrete. They may be employed in the analysis of structures of any material which can be connected rigidly at joints or connections between structural members.

(Diagrams are on the following pages) Readers are referred to the JOURNAL *for May 1930 for discussion which may develop. Such discussion should reach the Secretary by April 1, 1930.*

DIAGRAM 1 (SEE OPPOSITE PAGE BELOW)—CONTINUOUS BEAM OF TWO EQUAL SPANS. CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT AND SHEAR.

KEYS TO SYMBOLS:

w = Uniformly distributed D. L. and L. L. combined, in lbs. per lin. ft., with L. L. placed in most Unfavorable position.

L = Clear span length, in feet.

◔ = Curves for D. L. only, or for full L. L. on all spans.

① = Curves for case where L. L. = D. L.

② = Curves for case where L. L. = 2 times D. L.

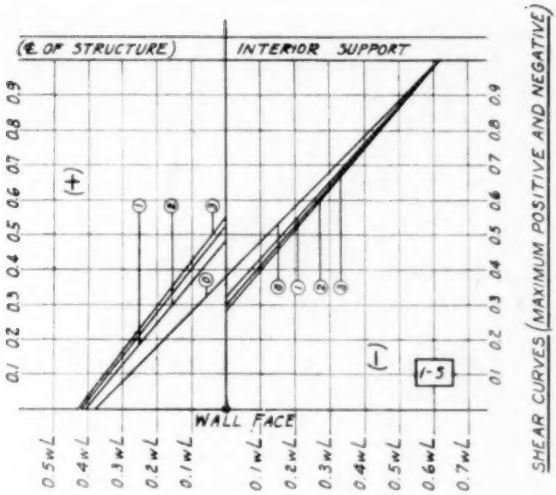
③ = Curves for case where L. L. = 3 times D. L.

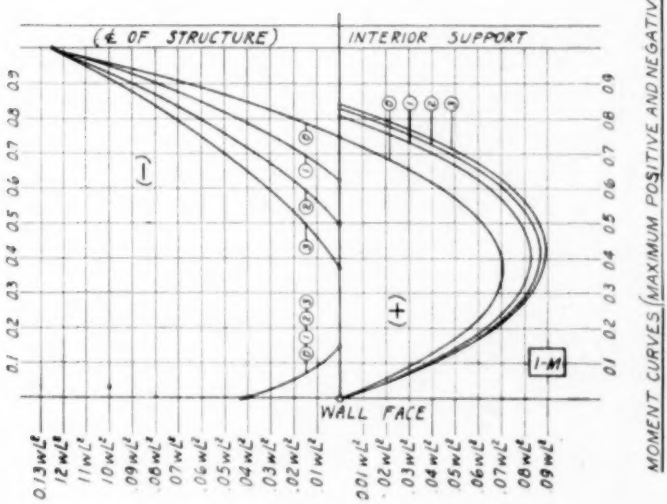
NOTES:

(1) Moments are in ft.-lbs. and shears are in lbs.

(2) For other ratios between L. L. and D. L., interpolate between curves shown.

(3) For unequal spans, see text.


(℄ OF STRUCTURE)
INTERIOR SUPPORT
WALL FACE
(+)
(−)
1-S
SHEAR CURVES (MAXIMUM POSITIVE AND NEGATIVE)


(℄ OF STRUCTURE)
INTERIOR SUPPORT
WALL FACE
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MOMENT CURVES (MAXIMUM POSITIVE AND NEGATIVE)

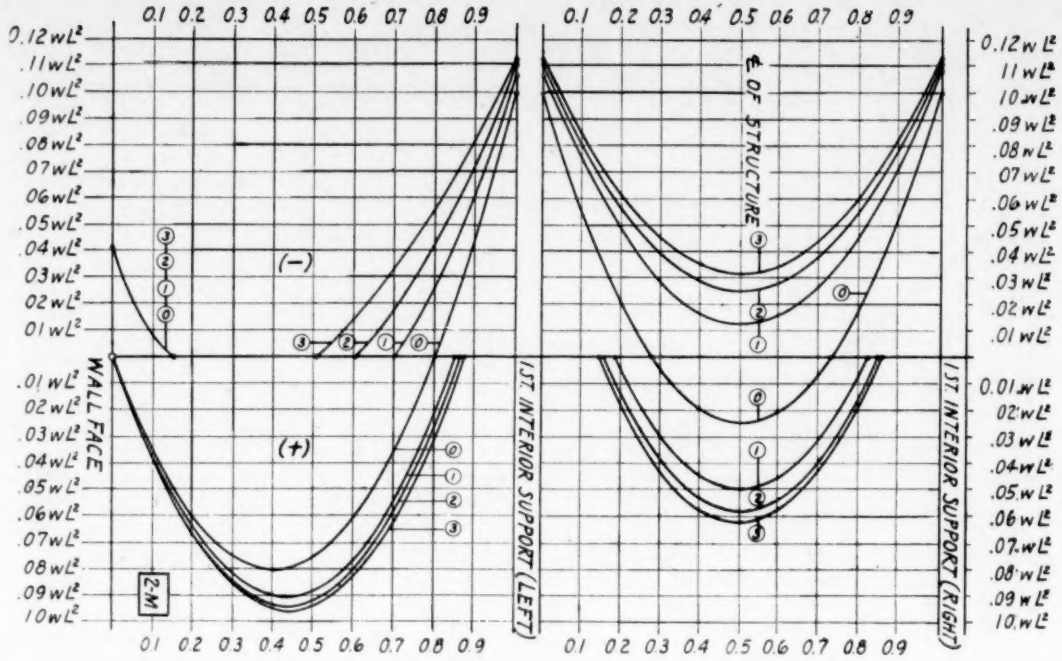


DIAGRAM 2—CONTINUOUS BEAM OF THREE EQUAL SPANS.
PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

KEY TO SYMBOLS:

w = Uniformly distributed D. L. and L. L. combined, in lbs. per lin. ft., with L. L. placed in most unfavorable position.

L = Clear span length, in feet.

③ = Curves for D. L. only, or for full L. L. on all spans.

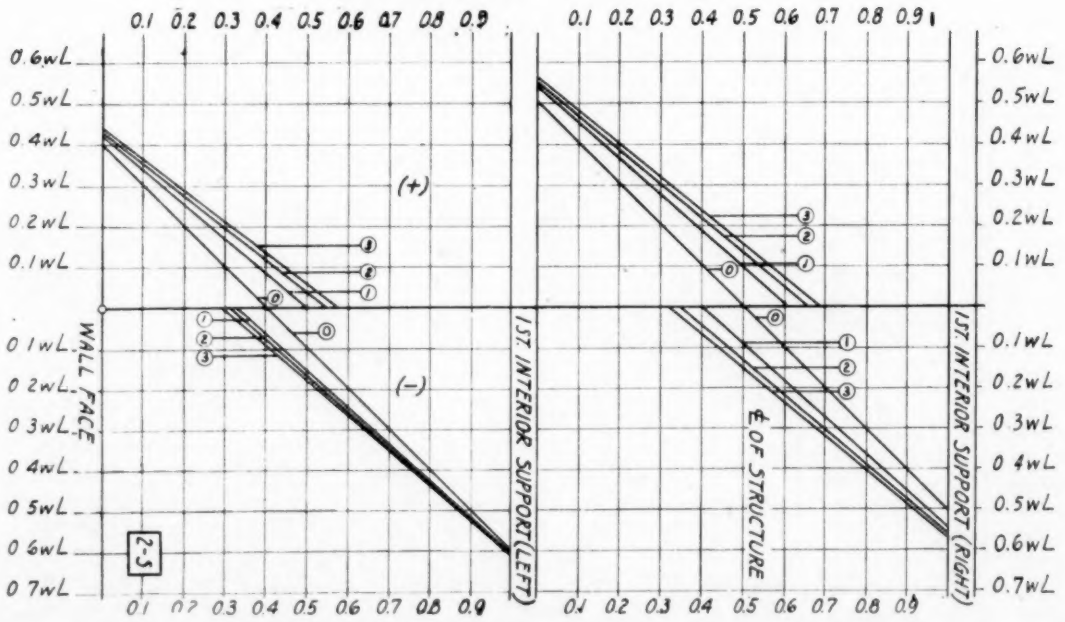
① = Curves for case where L. L. = D. L.

② = Curves for case where L. L. = 2 times D. L.

③ = Curves for case where L. L. = 3 times D. L.

NOTES:

(1) Moments are in ft.-lbs. and shears are in lbs.
(2) For other ratios between L. L. and D. L., interpolate between curves shown.
(3) For unequal spans, see text.



PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

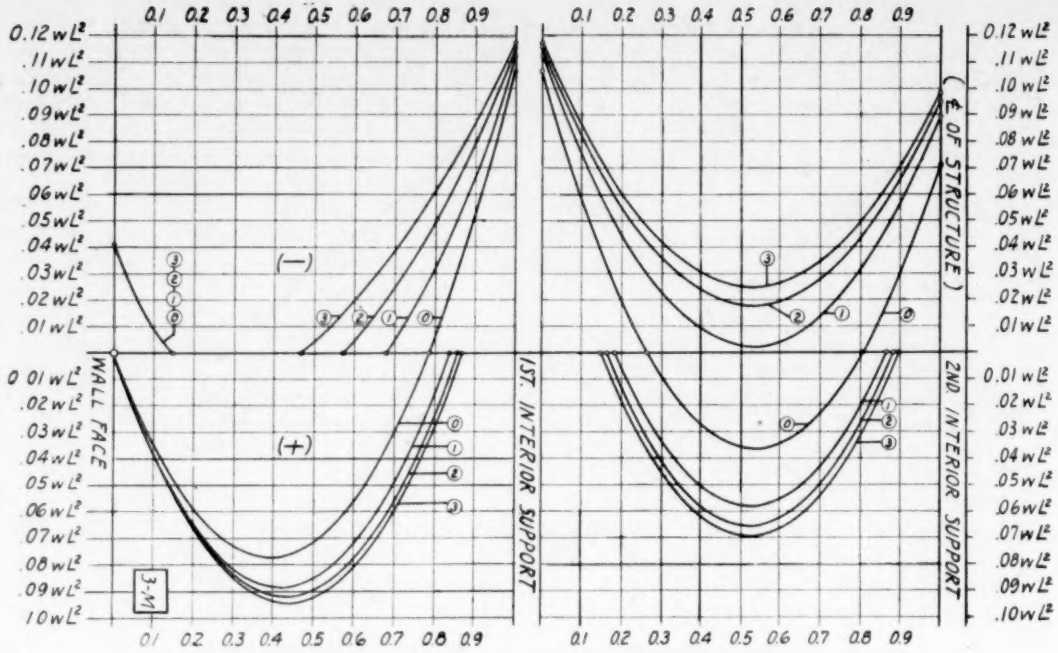


DIAGRAM 3—CONTINUOUS BEAM OF FOUR EQUAL SPANS.
PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

KEY TO SYMBOLS:

w = Uniformly distributed D. L. and L. L. combined, in lbs. per lin. ft., with L. L. placed in most unfavorable position.

L = Clear span length, in feet.

⓪ = Curves for D. L. only, or for full L. L. on all spans.

① = Curves for case where L. L. = D. L.

② = Curves for case where L. L. = 2 times D. L.

③ = Curves for case where L. L. = 3 times D. L.

NOTES:

(1) Moments are in ft.-lbs. and shears are in lbs.
(2) For other ratios between L. L. and D. L., interpolate between curves shown.
(3) For unequal spans, see text.

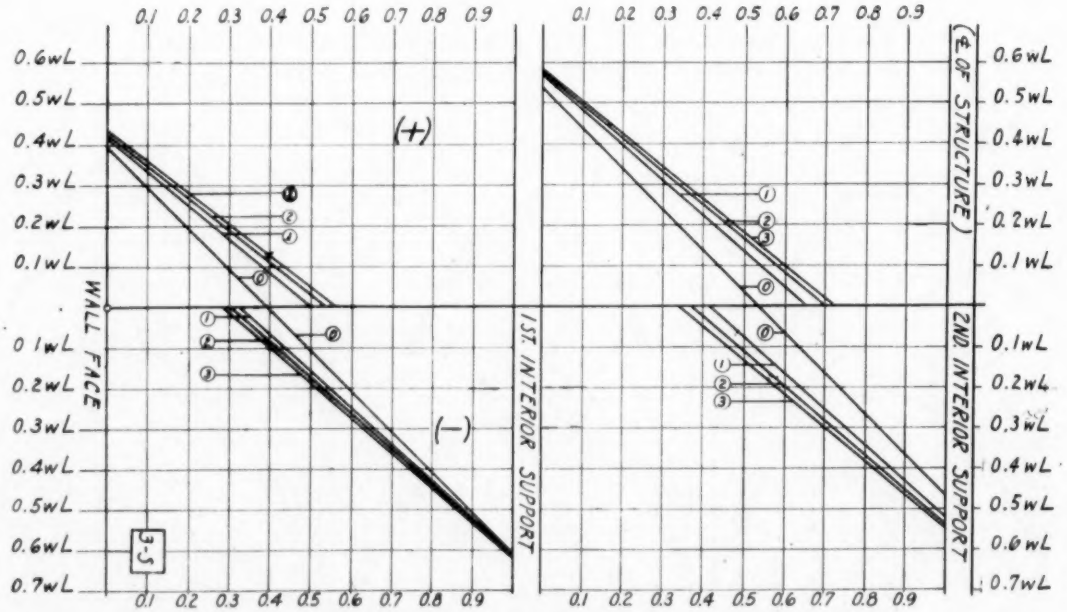


DIAGRAM 3—CONTINUOUS BEAM OF FOUR EQUAL SPANS.
PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

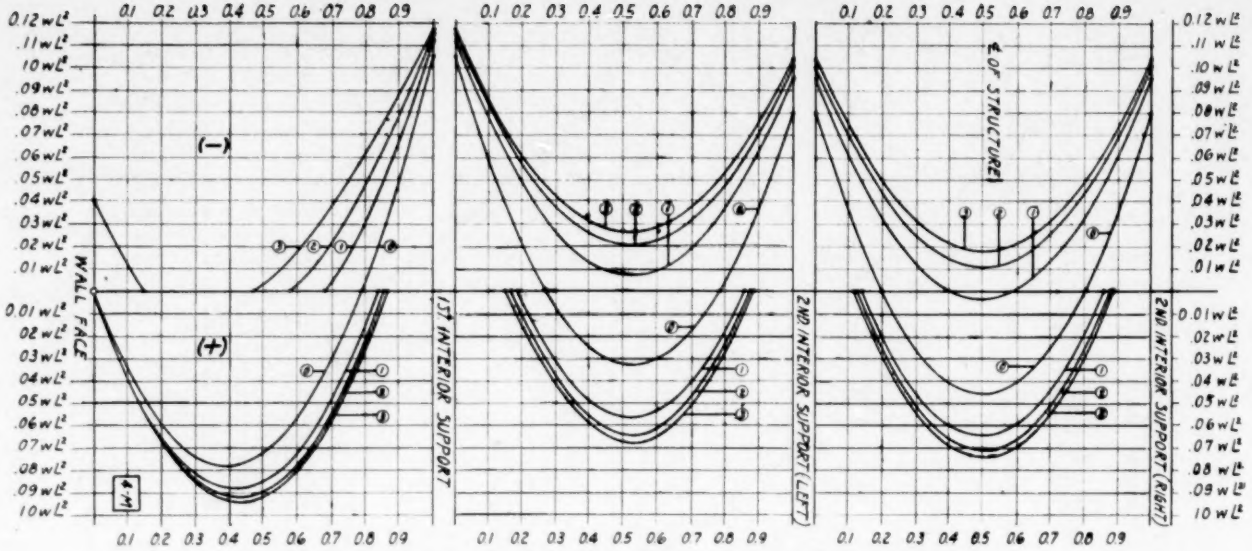


DIAGRAM 4—CONTINUOUS BEAM OF FIVE EQUAL SPANS.
PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

KEY TO SYMBOLS:

w = Uniformly distributed D. L. and L. L. combined, in lbs. per lin. ft., with L. L. placed in most unfavorable position.

L = Clear span length, in feet.

⑤ = Curves for D. L. only, or for full L. L. on all spans.

① = Curves for case where L. L. = D. L.

② = Curves for case where L. L. = 2 times D. L.

③ = Curves for case where L. L. = 3 times D. L.

NOTES:

(1) Moments are in ft.-lbs. and shears are in lbs.

For other ratios between L. L. and D. L., interpolate between curves shown.

(3) For unequal spans, see text.

(4) For more than 5 spans, the moments, shears and points of inflection for the additional supports and spans may be assumed at the above values for the middle span and the second interior support.

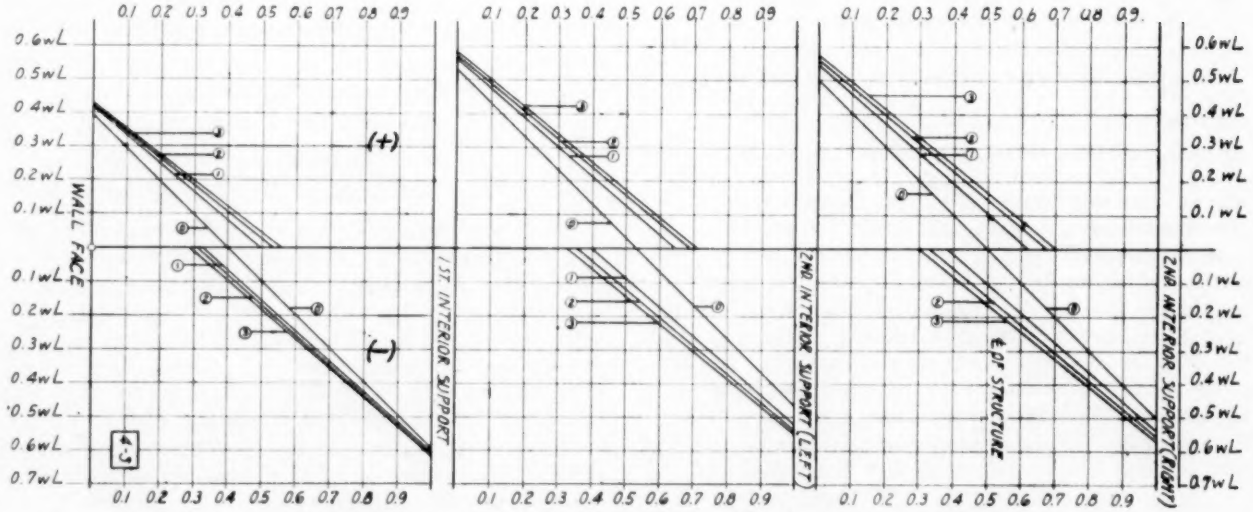


DIAGRAM 4—CONTINUOUS BEAM OF FIVE EQUAL SPANS.
PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

NOTES FOR DIAGRAMS 5 TO 18

KEY TO SYMBOLS:

w = Uniformly distributed D. L. and L. L. combined, in lbs. per lin. ft., with L. L. placed in most unfavorable position.

L = Clear span length in feet.

⓪ = Curves for D. L. only, or for full L. L. on all spans.

① = Curves for case where L. L. = D. L.

② = Curves for case where L. L. = 2 times D. L.

③ = Curves for case where L. L. = 3 times D. L.

$$\frac{I}{H} = \frac{\text{Moment of Inertia of column section}}{\text{Free length of column}}$$

$$\frac{I}{L} = \frac{\text{Moment of Inertia of girder section}}{\text{Clear span of girder}}$$

NOTES:

(1) Moments are in ft.-lbs. and shears are in lbs.

(2) For other ratios between L. L. and D. L., interpolate between curves shown.

(3) For additional interior spans, the moments, shears and points of inflection may be assumed at the above values for the third span and the second interior support.

(4) Column moments are in terms of wL^2 and drawn to same scale as girder moments, and are measured from the face of support as the zero axis.

(5) Formulas for Moment of Inertia: For reinforced concrete columns, the formula $I = I_c\ (1 + np)$ may be used, in which I_c is the Moment of Inertia of the gross area of concrete, n is the modulus ratio between steel and concrete, and p is the ratio of cross sectional area of vertical steel to gross area of concrete.

For reinforced concrete beams or girders, either the formula $I = 5/8 \cdot \frac{Md}{f_s} \cdot n$ or $I = \frac{35}{44} \cdot Ad_s^2 n$ may be used, in which M is the resisting moment of the girder, d is the distance from the compression surface to the center of the tension reinforcement, f_s is the unit stress in tension steel and A_s is the area of tension steel.

For columns and girders of other structural materials, use formulas in standard text books.

(6) Column moments are shown for a single column on the basis of equal moments in the column above and below the girders.

If the $\frac{I}{H}$ for the two columns differs, the sum of the moments for the upper and lower columns will be equal to the sum of the column moments obtained from the diagram, and the actual moment taken by either the upper or lower column will be in direct proportion to the sum as the $\frac{I}{H}$ value for the upper or lower column is to the sum of the two $\frac{I}{H}$ values. Thus if $\frac{I}{H}$ is 36 for the upper column and 54 for the lower, the upper column will take 36/90 and the lower column 54/90 of the sum of the moments for the two columns. Where a column occurs on one side of girder only (either below, as in ordinary roof framing, or above) the sum of the moments for two columns must be taken by the single column. Such a condition is in fact shown on Diagrams 5, 7, 9, 12, 14 and 16. Shears in columns may be obtained by dividing the maximum bending moment by the distance from the section where this B. M. occurs to the point of contra-flexure of the column.

(7) In using these Diagrams for interior spans, the ratio of the sum of the I/H for the columns above and below any interior joint to the sum of the I/L for the two girders must be used. For a roof frame the I/H of the single column below *is* the sum of the I/H for the two columns normally present at a joint. This latter case is the condition shown on Diagrams 5, 7, 9, 12, 14 and 16.

(8) In these Diagrams all columns are assumed to be equally stiff and all girders are assumed to be equally stiff. See text for the procedure to follow when this condition is not true.

(9) For end spans a modification must be made in using the Diagrams, since only one girder frames into the end column. For this case the ratio of the sum of the I/H for the column above and below the end joint to *twice* the I/L for the girder in the end span must be used. In the case of end spans of roof frames the ratio to be used is that of the I/H of the single column present to *twice* the I/L of the end girder.

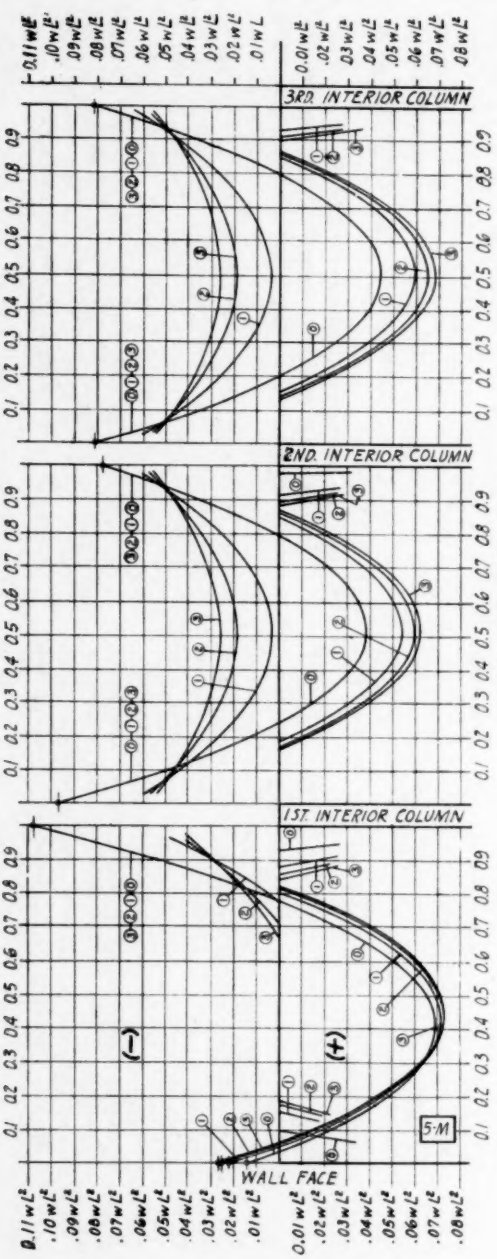


DIAGRAM 5—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 1/10 OF SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

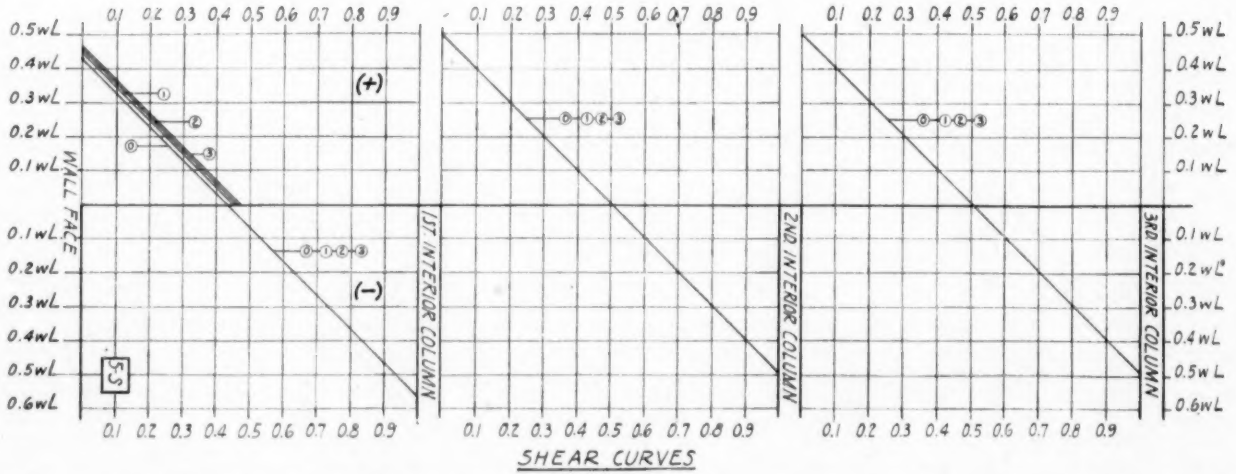


DIAGRAM 5—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 1/10 OF SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

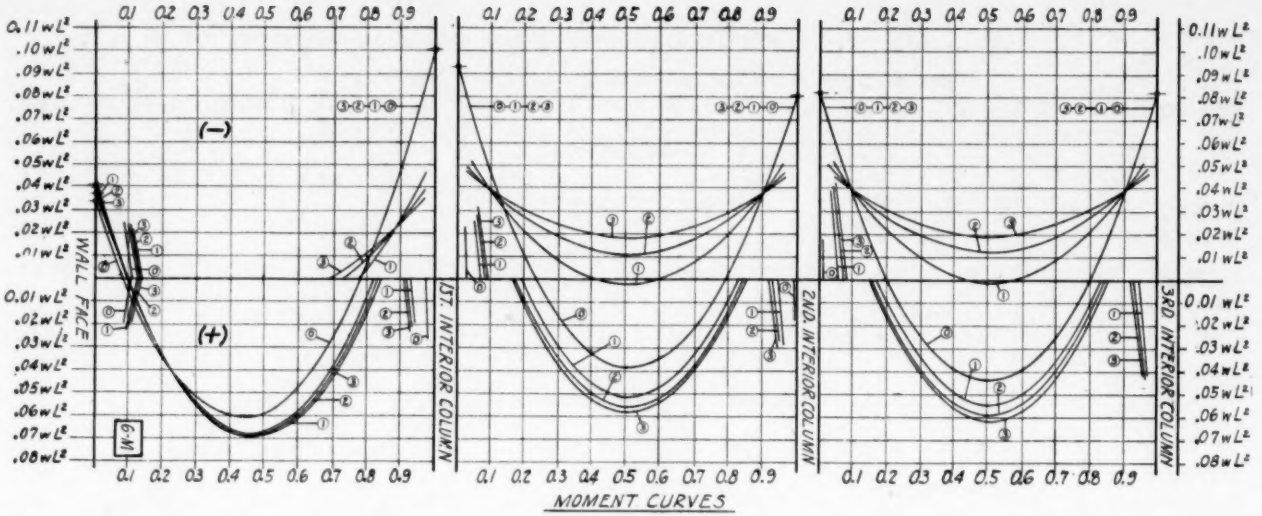


DIAGRAM 6—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 1/5 OF SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

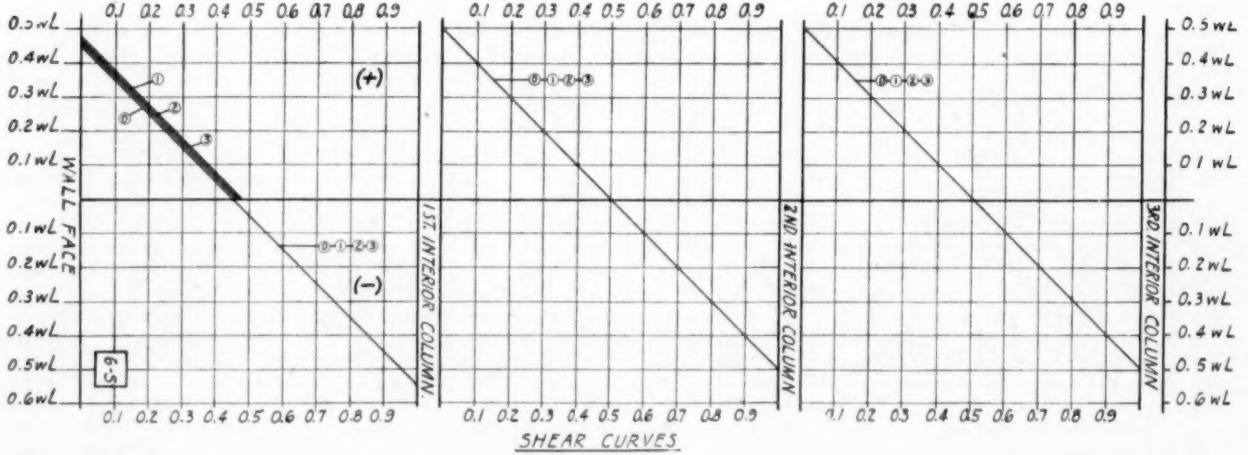


DIAGRAM 6—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 1/5 OF SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

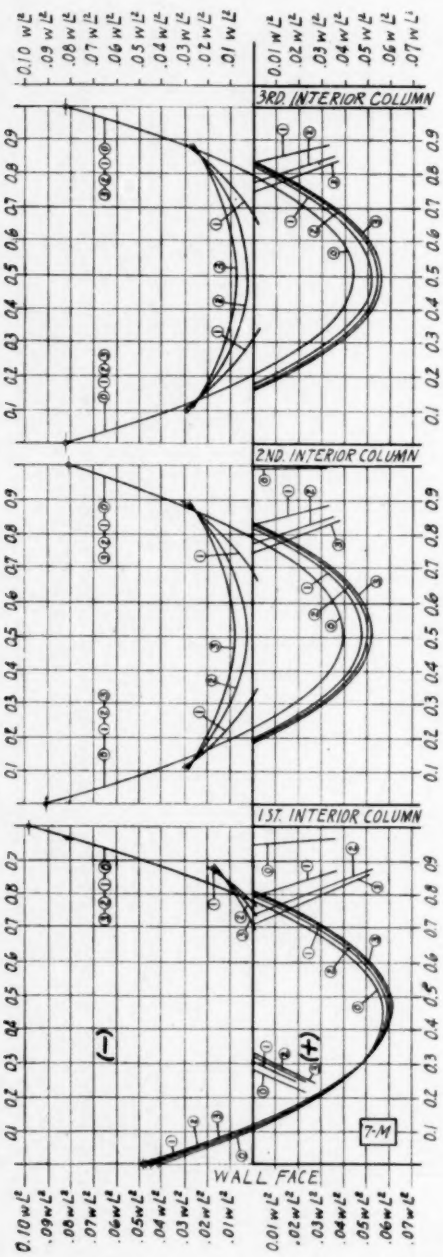


DIAGRAM 7—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 1/2 OF SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

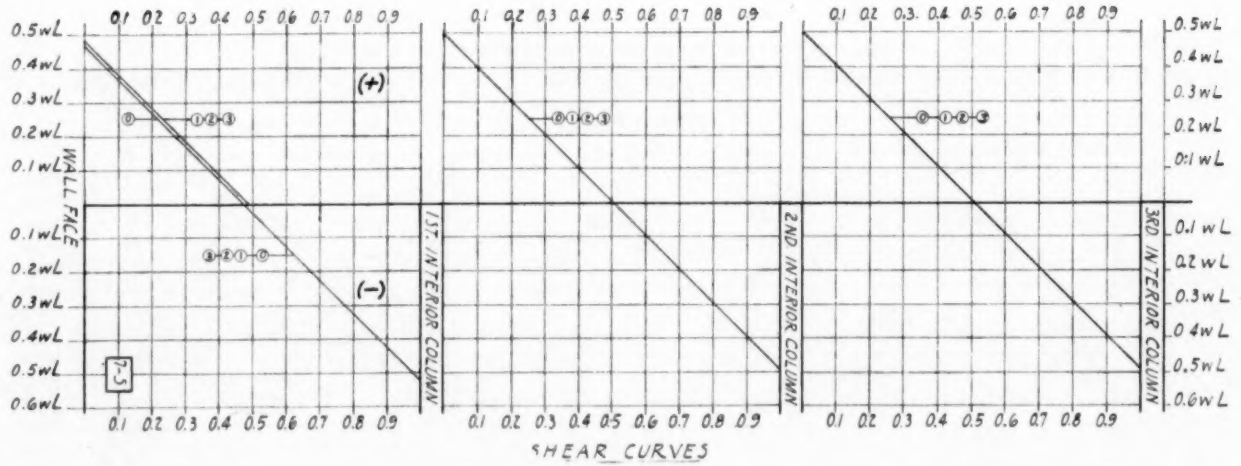


DIAGRAM 7—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 1/2 OF SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

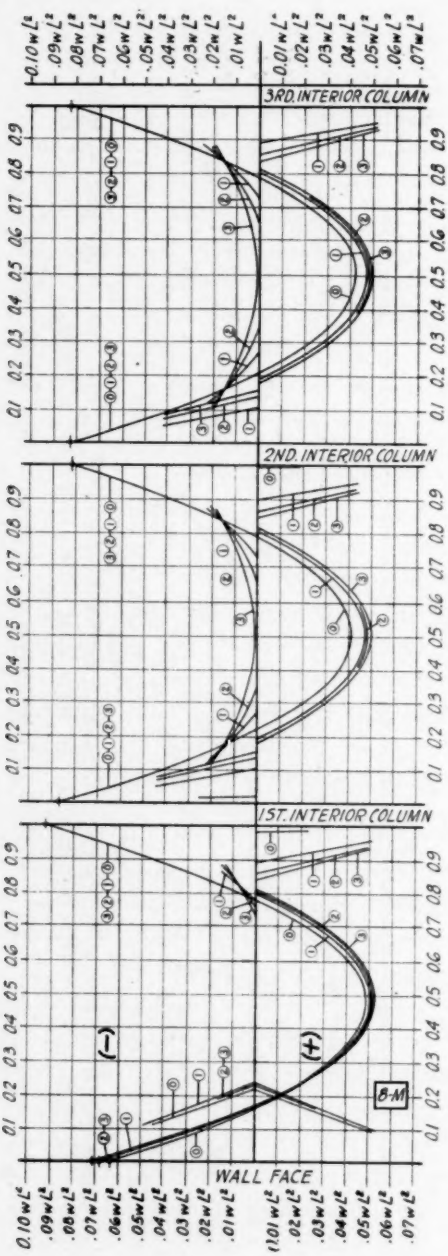


DIAGRAM 8—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO SUM OF I/L OF GIRDERS.

PART ONE—CURVE FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

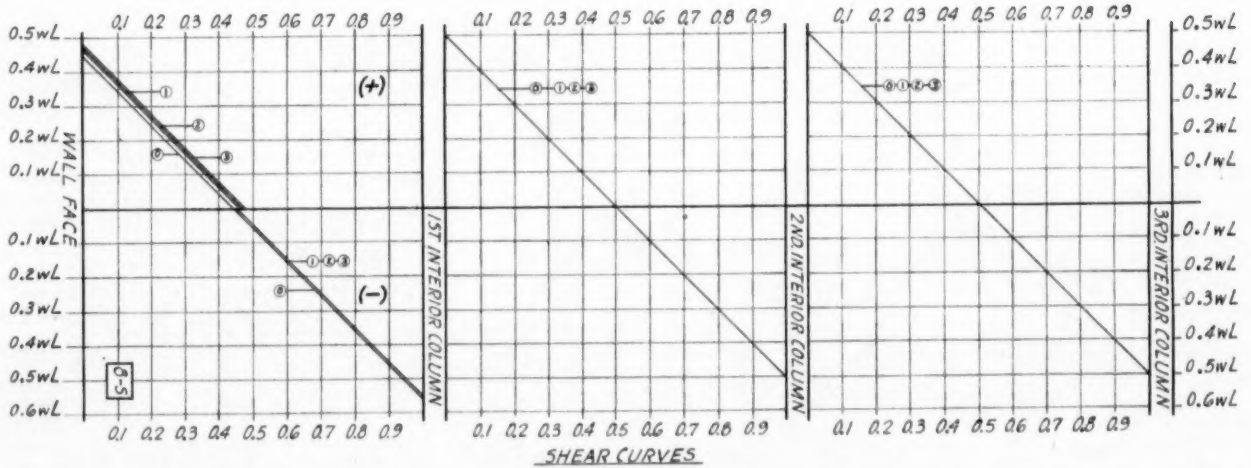


DIAGRAM 8—RIGID BUILDING FRAME—WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

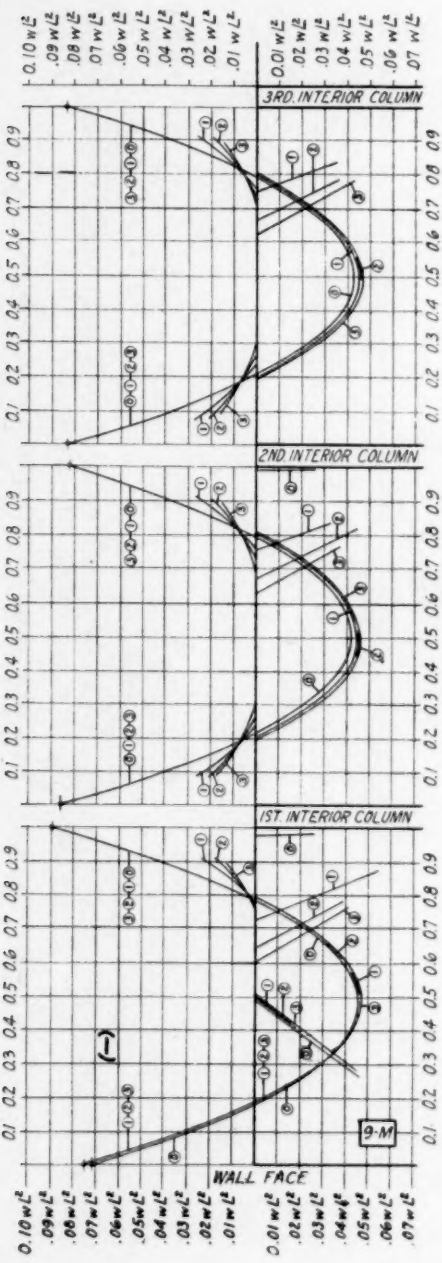


DIAGRAM 9—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 2½ TIMES SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

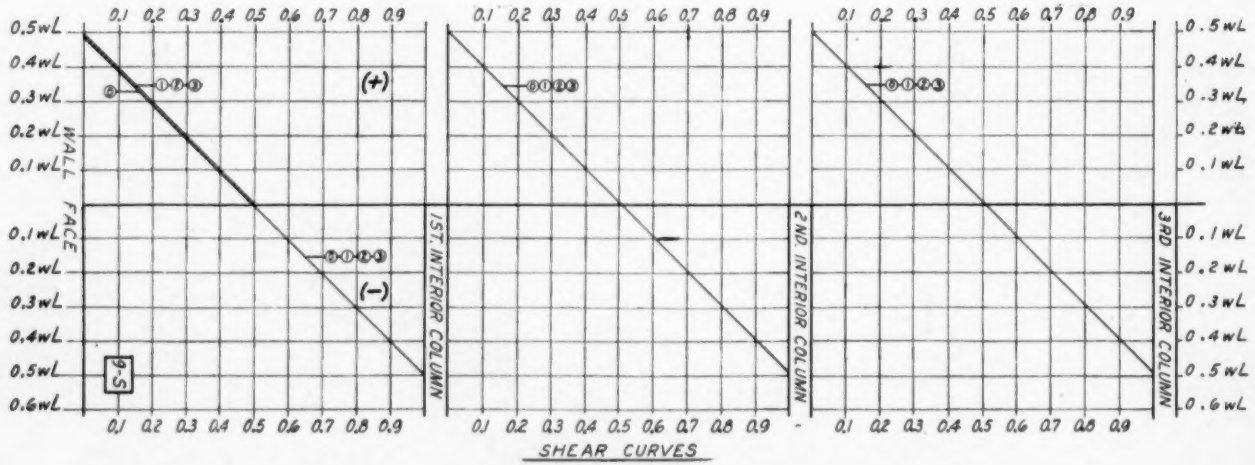


DIAGRAM 9—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 2½ TIMES SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

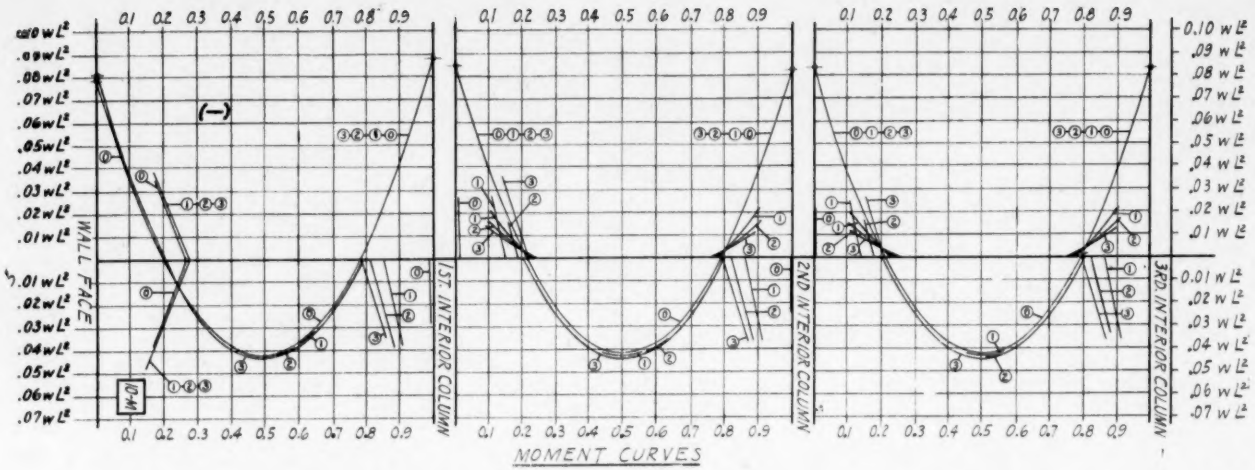


DIAGRAM 10—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 5 TIMES I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

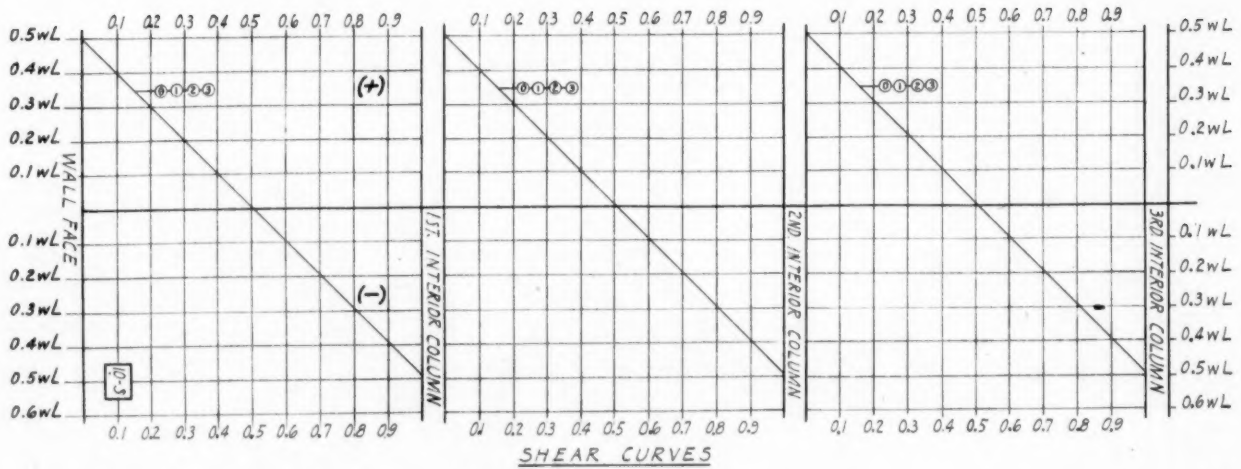


DIAGRAM 10—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 5 TIMES I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

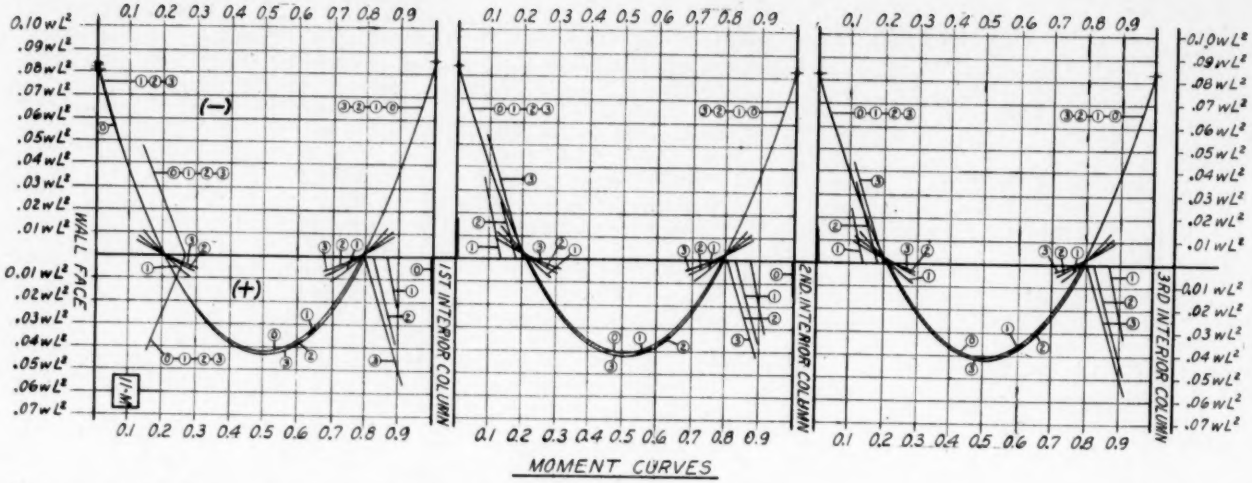


DIAGRAM 11—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 10 TIMES I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

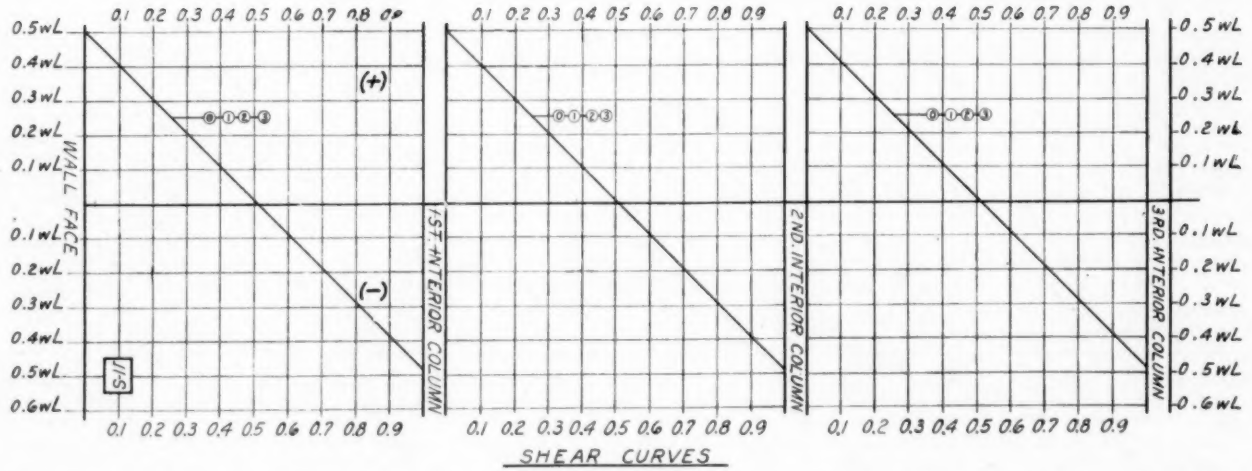


DIAGRAM 11—RIGID BUILDING FRAME, WALL-END OF GIRDER FRAMING INTO WALL COLUMN—SUM OF I/H OF COLUMNS EQUAL TO 10 TIMES I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

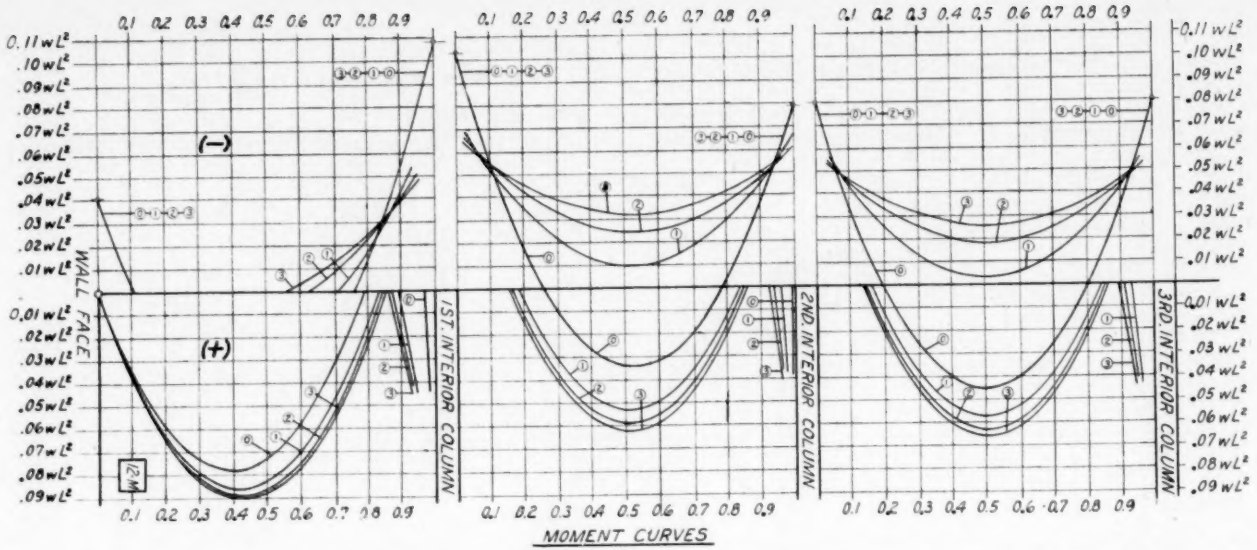


DIAGRAM 12—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 1/10 OF SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

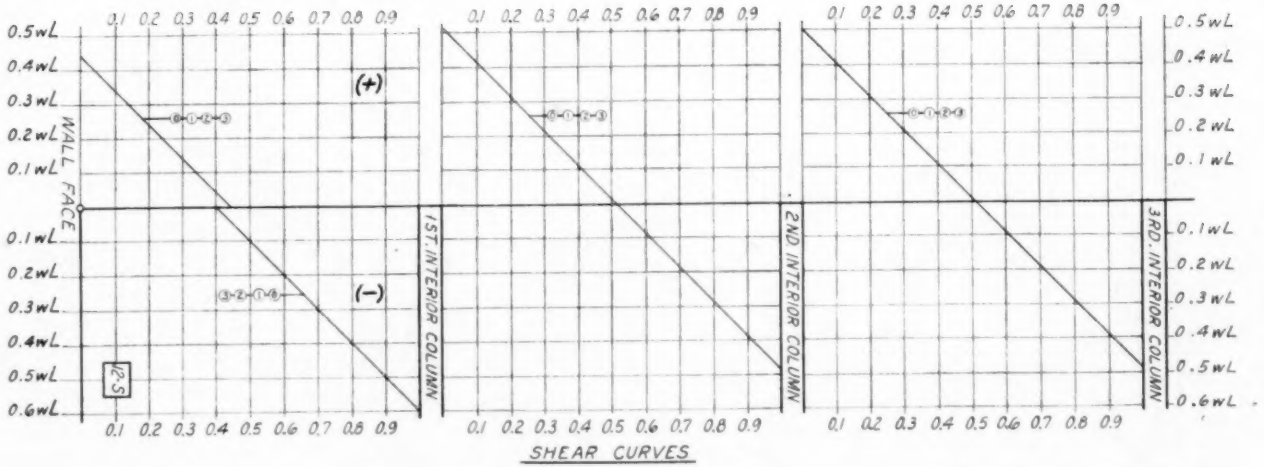


DIAGRAM 12—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 1/10 OF SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

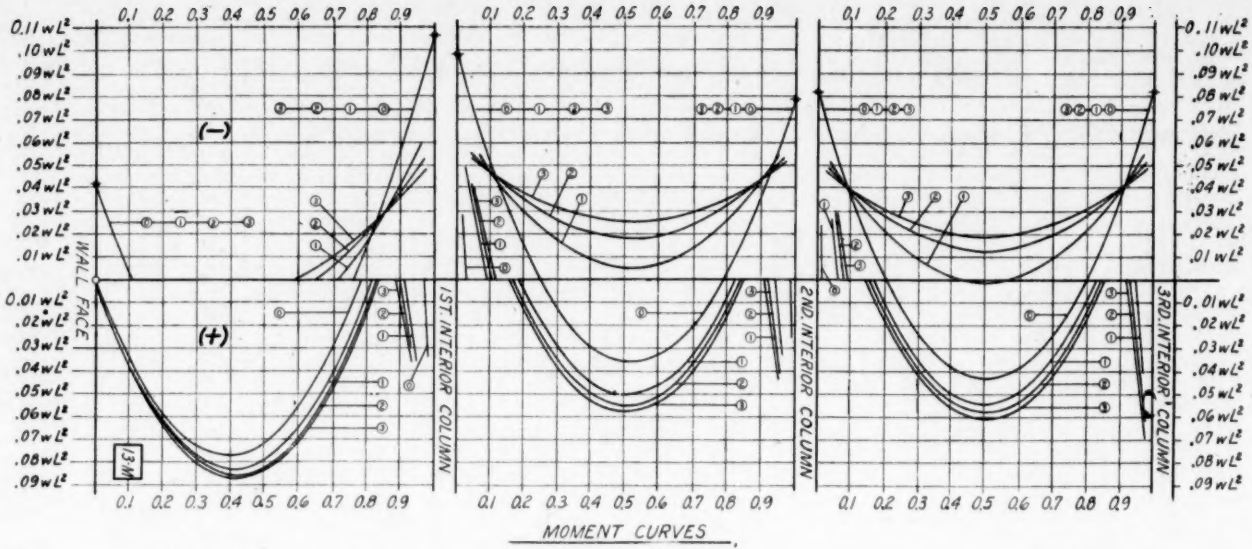


DIAGRAM 13—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 1/5 OF SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

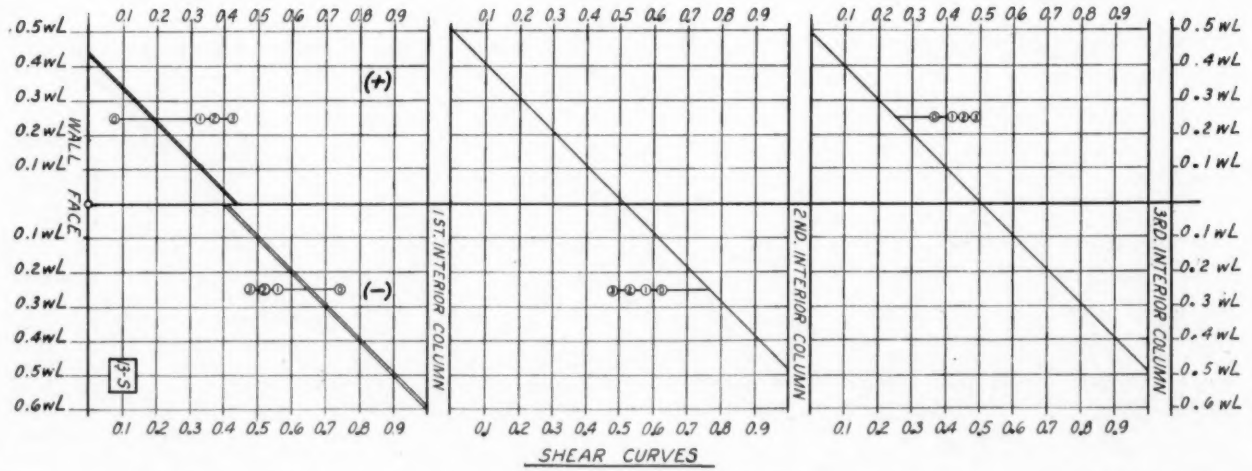


DIAGRAM 13—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 1/5 OF SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

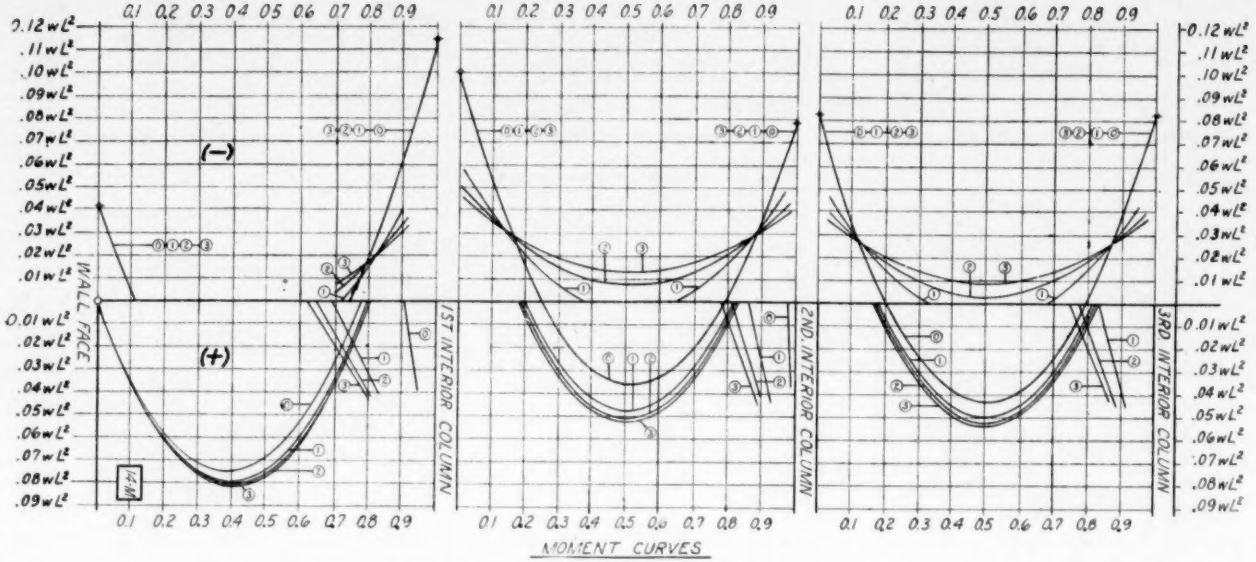


DIAGRAM 14—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 1/2 OF SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

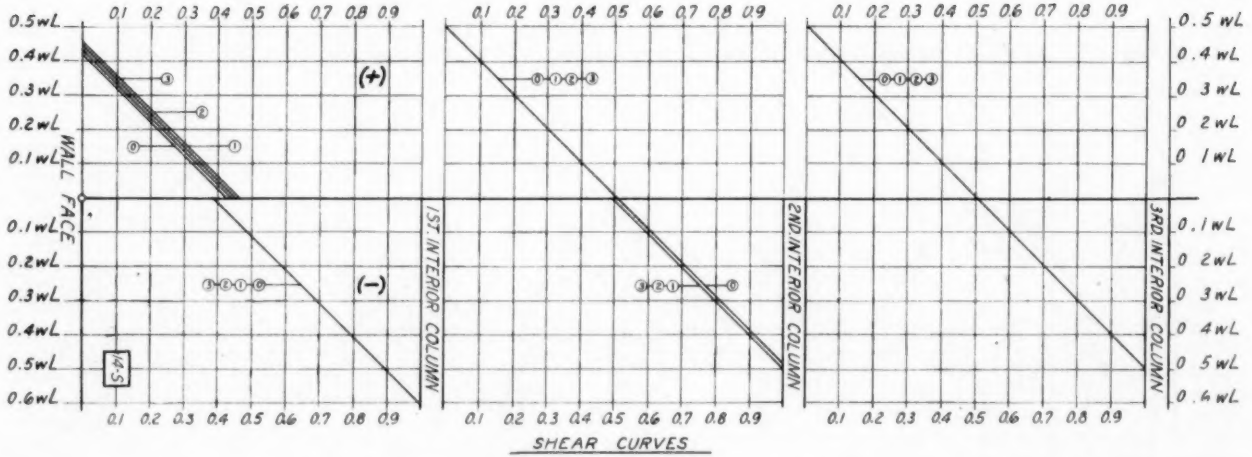


DIAGRAM 14—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 1/2 OF SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

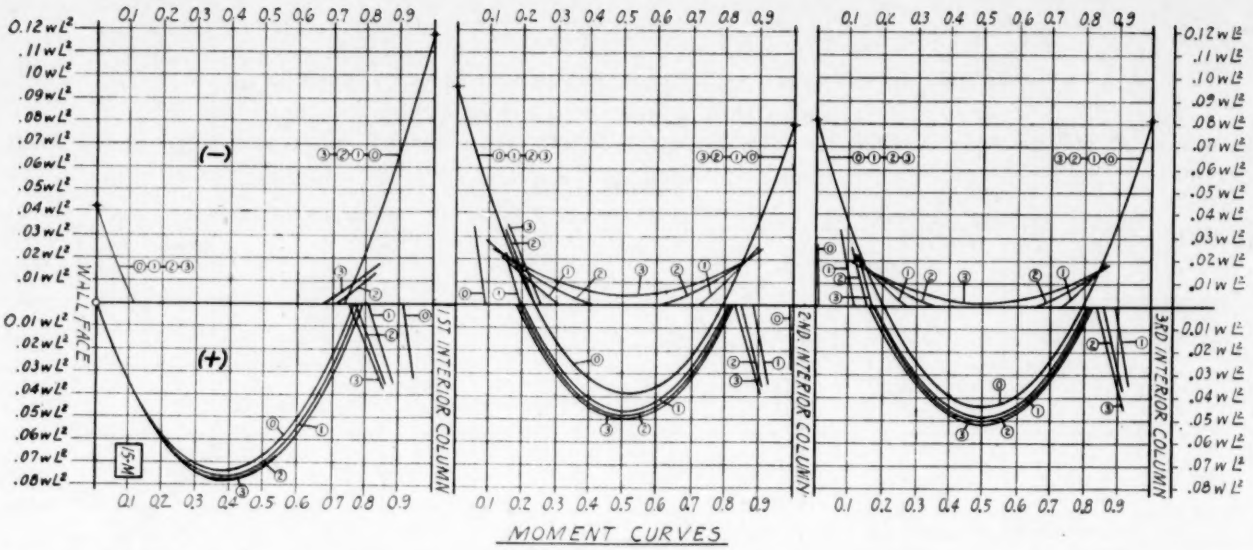


DIAGRAM 15—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

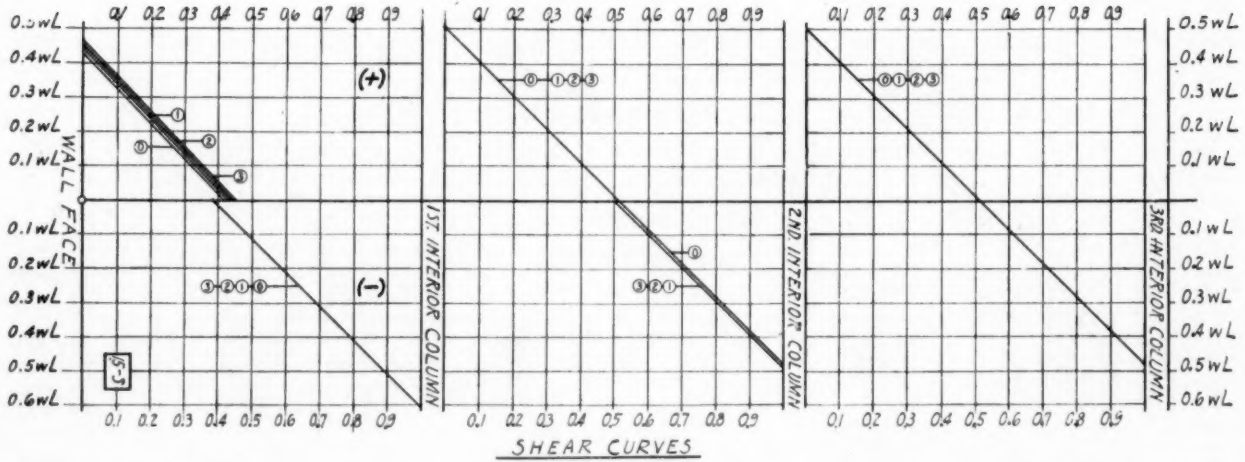


DIAGRAM 15—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

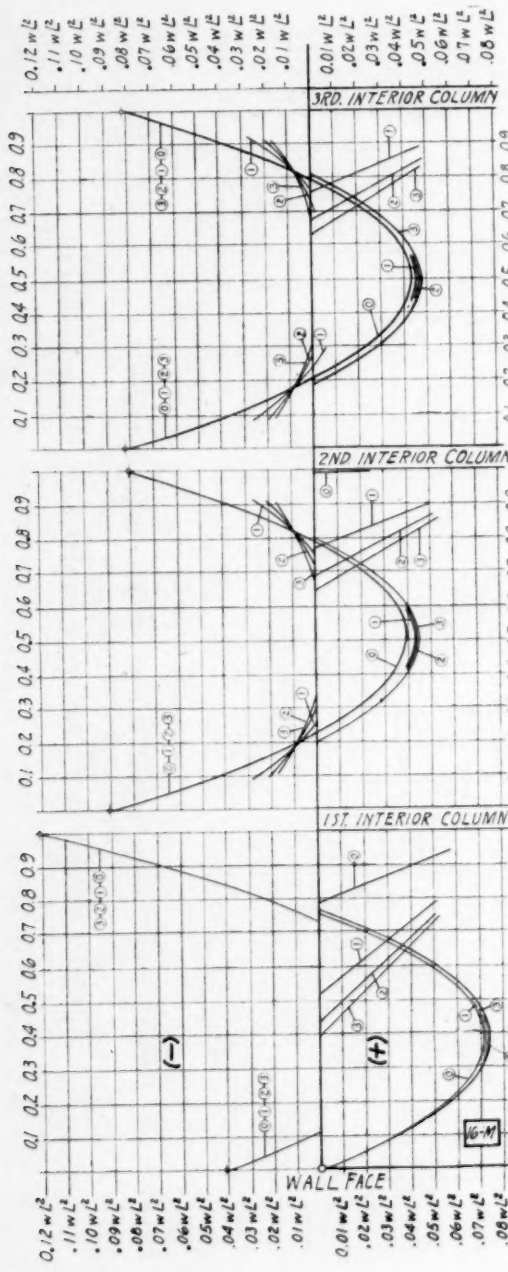


DIAGRAM 16—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 2½ TIMES SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

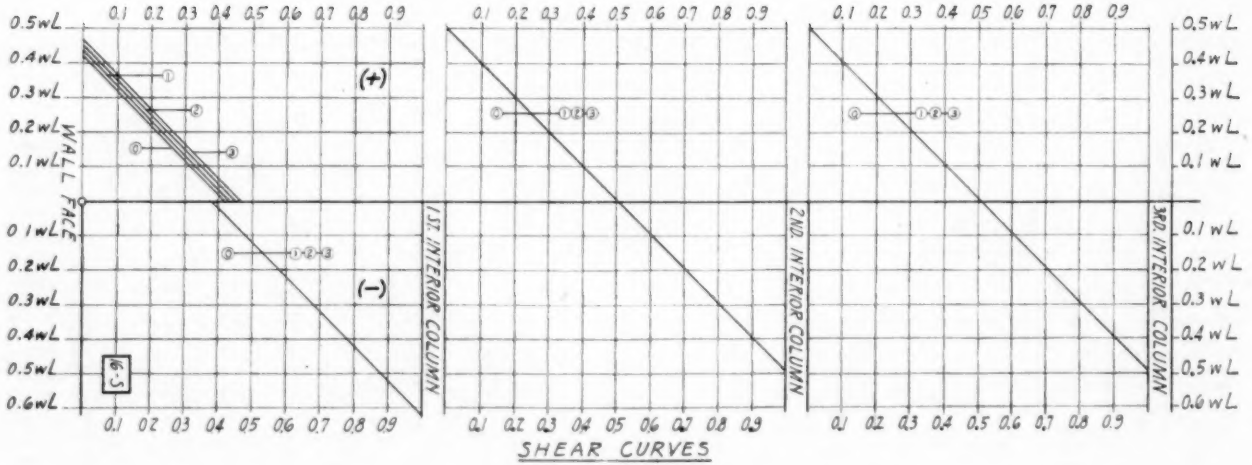


DIAGRAM 16—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 2½ TIMES SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

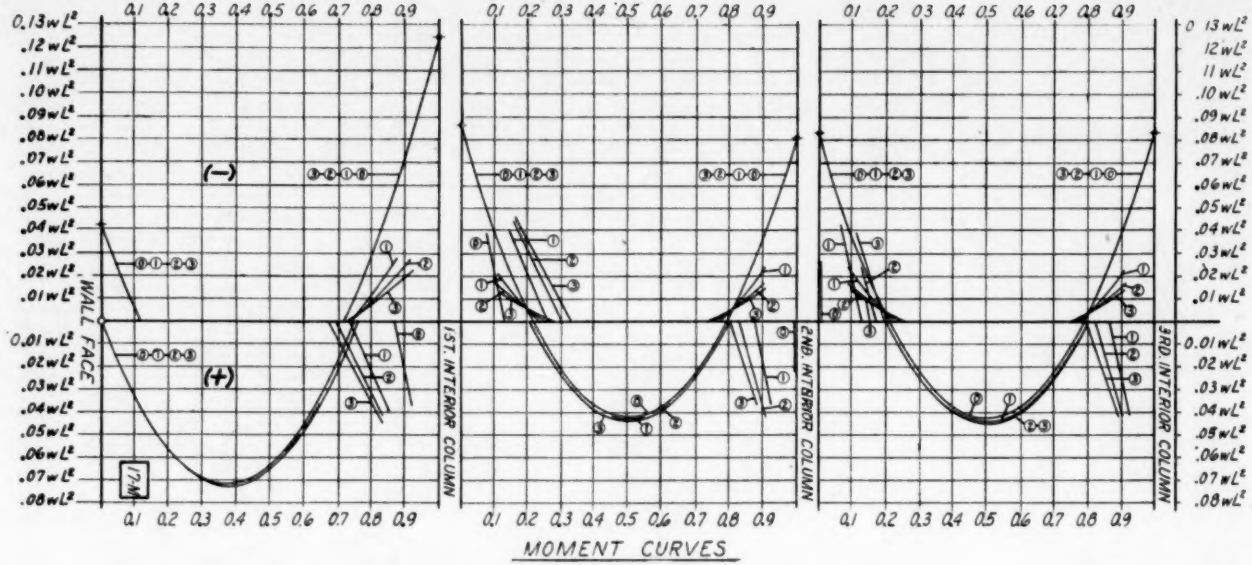


DIAGRAM 17—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 5 TIMES SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

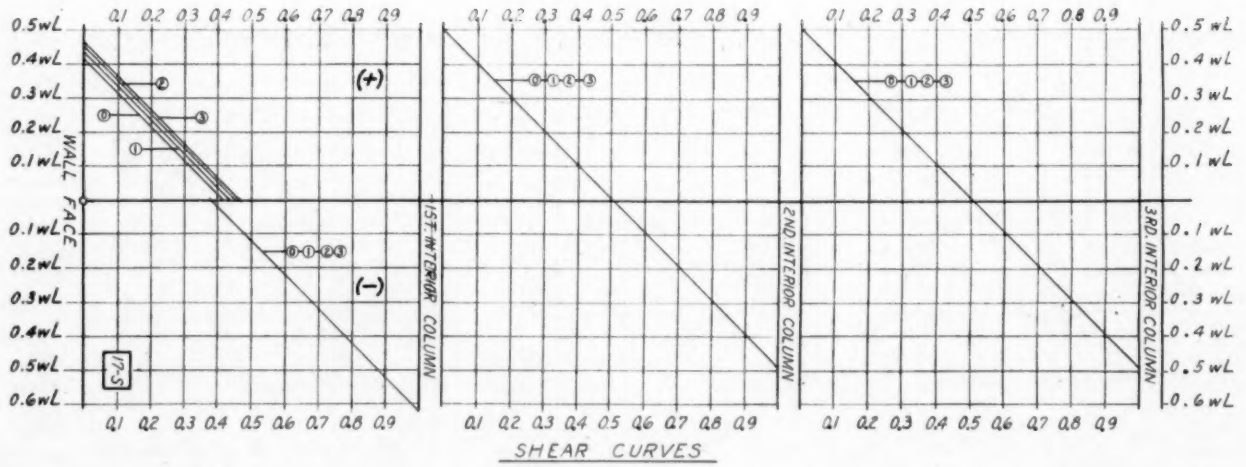


DIAGRAM 17—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 5 TIMES SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

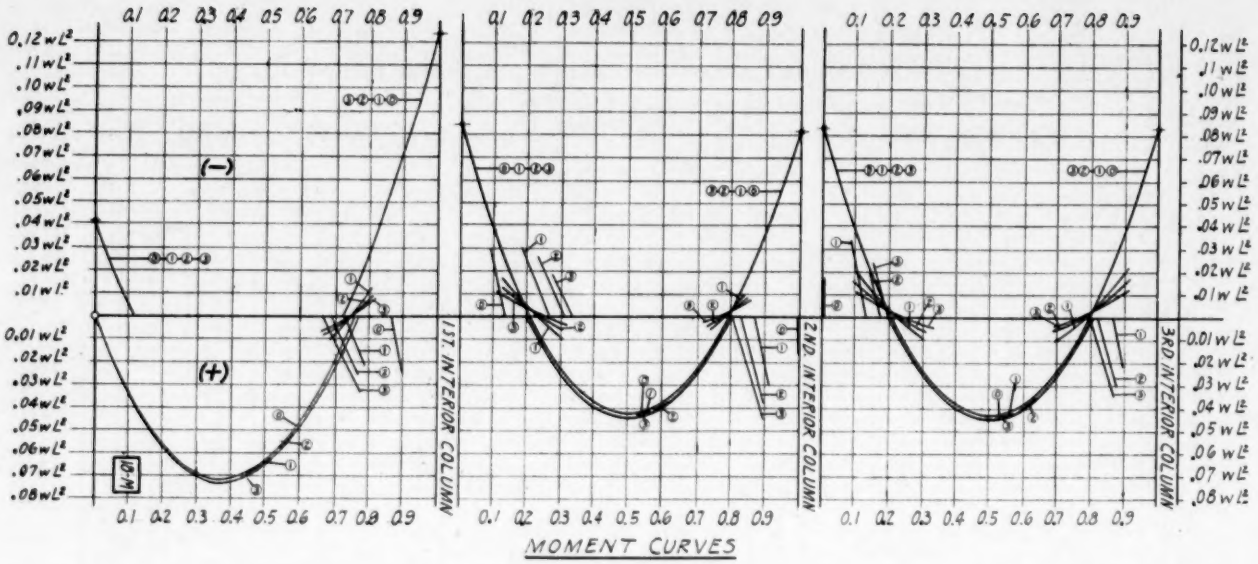


DIAGRAM 18—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 10 TIMES SUM OF I/L OF GIRDERS.

PART ONE—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE MOMENT.

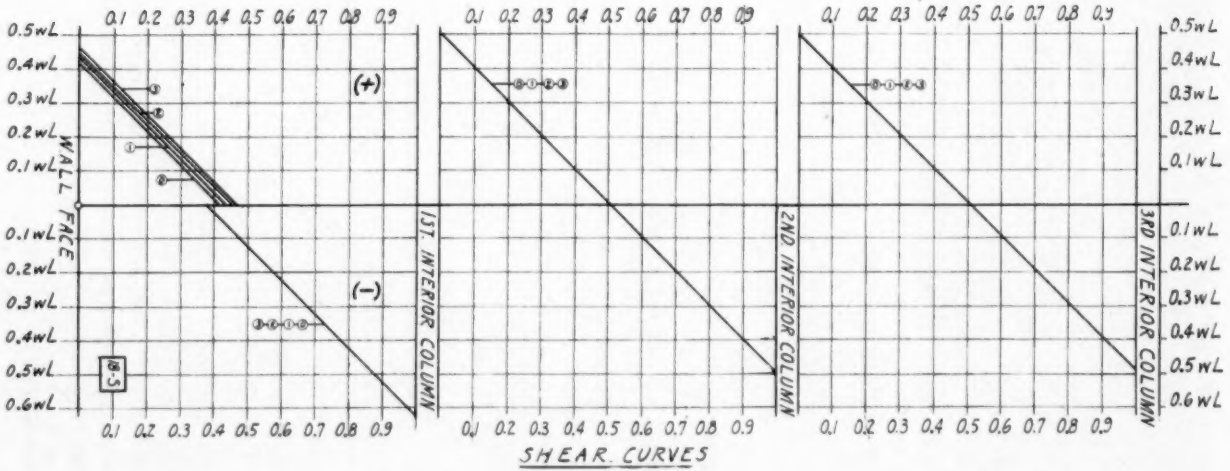


DIAGRAM 18—RIGID BUILDING FRAME, WALL-END OF GIRDER FREELY SUPPORTED—SUM OF I/H OF COLUMNS EQUAL TO 10 TIMES SUM OF I/L OF GIRDERS.

PART TWO—CURVES FOR MAXIMUM POSITIVE AND NEGATIVE SHEAR.

Observation of An Exposed Reinforced Concrete Beam

BY W. I. FREEL*

In March 1928, in the process of moving the Testing Materials Laboratory of Purdue University from its old home in the Mechanical Engineering building to the new wing of the Civil Engineering building recently constructed for it, a concrete test beam of the vintage of 1904 was unearthed. The beam had been tested to destruction in the laboratory and exposed at ground line to the showers of twenty-four summers and the freezes and thaws of twenty-four winters. At the center of the beam the rods were exposed in open cracks. The beam shows the deterioration of reinforcing steel embedded in cracked concrete and the perfect condition of the reinforcing steel embedded in the sound concrete a short distance away. See photographs.

The concrete was chipped from around the steel at the fractured portion of the beam, and the reinforcement exposed for a distance of eight inches along what had been the tension side of the beam. Nearby a section of the reinforcement encased in the sound part of the beam was exposed by chipping as shown in the photographs. The steel encased in the sound part of the beam was found to be smooth and without rust showing the mill scale tightly adhering to the bar. The steel embedded in the cracked concrete was found heavily coated with thick scales of rust which easily came loose from the bar exposing deep pits in the surface. Some rust scales were as thick as $\frac{3}{32}$ in. The steel was embedded one inch in the concrete.

Fortunately the beam could be identified by its markings as Beam A-5 of a series fabricated and tested by William Lucian Bridges and Joseph Warren Ernst for an undergraduate thesis in 1904. Some of the details of the history of the beam may be brought out by quotations from the undergraduate thesis above

*Instructor in Civil Engineering, Purdue University.

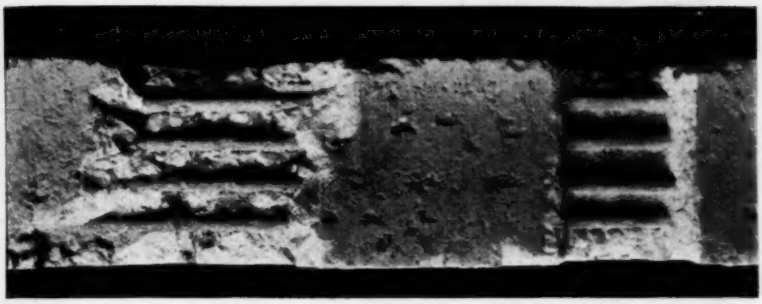


FIG. 1—RUST INCRUSTED STEEL EMBEDDED IN CRACKED CONCRETE AND STEEL IN PERFECT CONDITION IN SOUND CONCRETE A FEW INCHES AWAY.

mentioned, and from one other closely coordinated with it by Silas Lee Pierce and George Ulric Middleton of the same year.

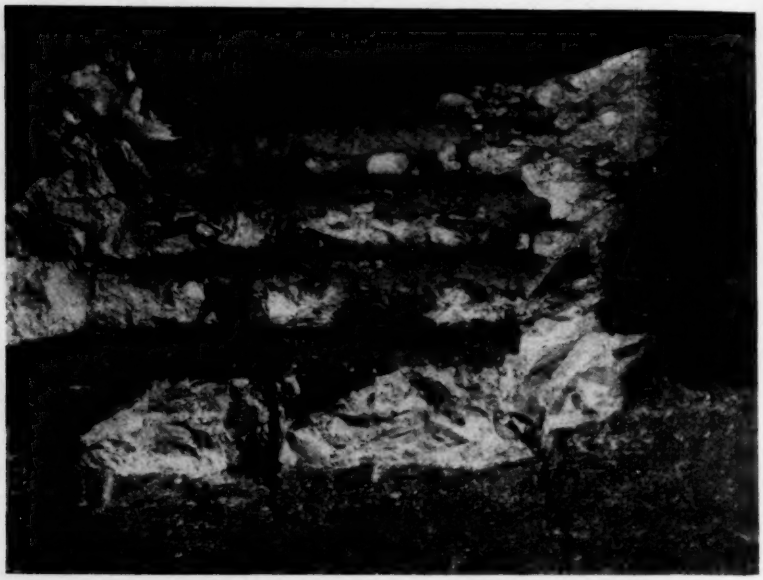
The object of this thesis is the determination of strength of reinforced concrete beams made in the proportions of one part cement, two parts sand, and four parts broken stone, with different reinforcements of steel and also to see how far these tests carry out the reliability of the theory of the strength of reinforced concrete beams as presented by Professor W. K. Hatt, of Purdue University, in "Tests of Reinforced Concrete Beams." (This paper appears in the Proceedings of the American Society for Testing Materials, Vol. II, 1902).

The cement used was the Giant brand of portland cement made by the American Cement Company.

The broken stone was limestone from Kankakee, Illinois, crusher run below one inch all passed through a one-inch sieve.

The sand was clean sharp pit sand obtained from the sand pit at Lafayette. It contained 33⅓ per cent voids and 37 per cent was retained on a No. 20 sieve.

The concrete out of which the beams were molded, was made in the proportion of one part cement, two parts a local sand, and four parts broken stone. The method of mixing was by volume. The cement and sand were thoroughly mixed together before the stone was added. The broken stone was sprinkled with water before being added to the cement and sand. After the sand and cement were mixed dry the water was applied; and mixed until the color of the sand and cement was uniform. Then the stone was added and the mass cut over six times until the stone was thoroughly covered with cement. The concrete was a medium wet concrete.



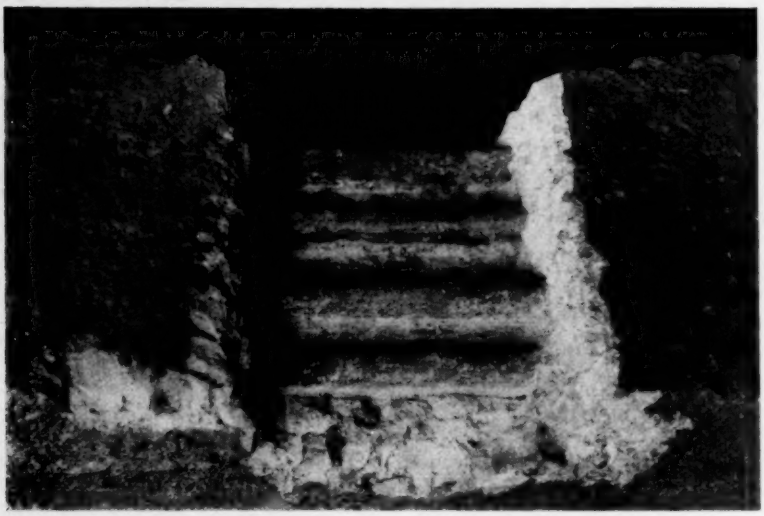


FIG. 2—ABOVE—A CLOSE-UP OF THE RUSTED STEEL SHOWN AT LEFT IN FIG. 1.

FIG. 3—BELOW—A VIEW OF THE STEEL IN PERFECT CONDITION.

The concrete resulting from this method of proportioning and mixing had an average strength at 80 days of 2075 lbs. per sq. in. in compression and an average strength of 286 lbs. per sq. in. in tension. An indication of the quality of cement used may be

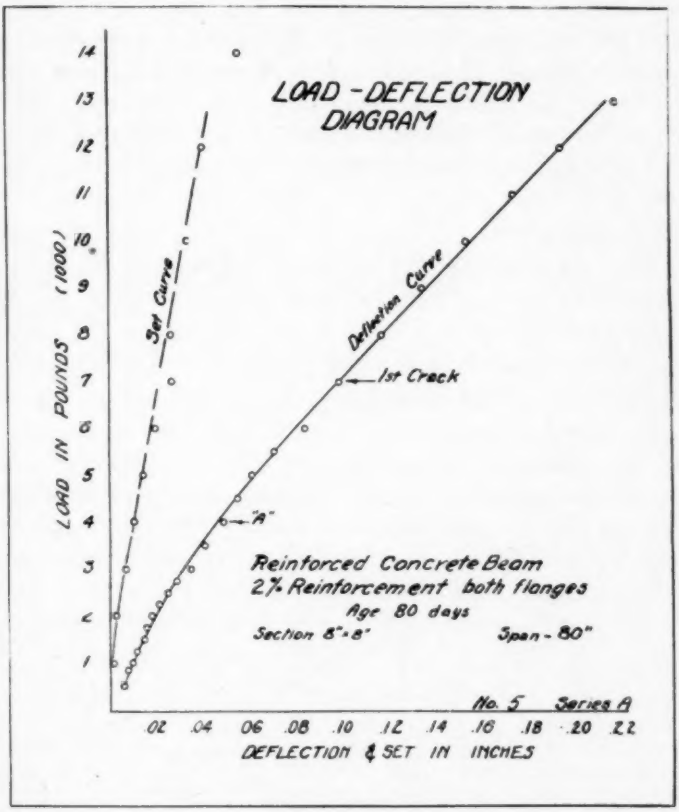


FIG. 4.

obtained by noting that the 1:3 mortar briquettes had strengths at 7 and 28 days respectively of 275 lbs. per sq. in. and 267 lbs. per sq. in.

When the concrete broke the stone split. On breaking away the concrete from the reinforcements it was found that the rods did not slip, but exceeded their elastic limit as was shown by the scaly appearance of the metal.

The load deflection diagram for the beam shows that the first crack opened when the 7000 lb. load was applied. This indicates that an external moment of 140,000 inch pounds was sufficient to crack the section now showing rusted steel. The external moment acting at any section is not altered by such cracks, therefore, it is reasonable to suppose that the external moment of 224,750 inch pounds, acting on the section at present showing no rusting of steel, did produce fine cracks which closed immediately after the load was removed. This observation may be taken as additional confirmation of the fact that such very fine cracks when drawn together again do not admit rust producing agencies.

Readers are referred to the JOURNAL *for May 1930, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1930.*

A Study Of Chapter II. "Tentative Building Regulations For Reinforced Concrete"*

DISCUSSION BY PHIL. J. MARKMANN

(*Author's closure, by letter, referring to Mr. Maney's letter, p. 826 Vol. 25.*) The correspondent's comparison of the "Chicago" and "Tentative Regulations" codes, for $n = 15$, $f'_c = 2000$, f_c in the Chicago code $= \frac{f_c'}{4}(1 + 37.5\ p')$, is a little confused (due probably to haste). The equation, bottom line of p. 827 was intended to read:

"Tentative Regulations" $\frac{P}{A_c} = 500\ (1 + 14p)\ (1 + 16p)$, and the comment on the two equations was intended to be: "and if in the former equation we make $p' = 0.427\ p$ — the two equations become the same." This amendment in formula and text following clears up the demonstration. The writer acting upon the explanation that eq. 22a has been devised to make allowance for the eventual increase of the steel stress as the result of "flow" of the concrete finds that for an assumed "flow" corresponding to an increase of the "design" $n = 15$ to an eventual $n = 60$.

	p					
	.01	.02	.03	.04	.05	.06
the "design" concrete stresses ..	580	660	740	820	900	980
decrease to.....	415	388	379	380	387	396
the "design" steel stresses.....	8700	9900	11100	12300	13500	14700
increase to.....	24900	23380	22740	22800	23320	23760

The rather high concrete "design" stresses for the higher steel ratios were objected to by the writer as exceeding conservative safe values. Limiting the "design" concrete stress to 740 (the formula value for $p = .03$) we would have

*Proceedings American Concrete Institute, Vol. 25, 1929, p. 818.

	p		
	.04	.05	.06
the "design" concrete stresses	740	740	740
decrease to	342	318	300
the "design" steel stresses	11100	11100	11100
increase to	20500	19100	18000

Thus, while the eventual steel stresses, for the assumed eventual $n = 60$ would be less than in the former case we would design with a more conservative concrete stress, viz. 740 lb.

Limiting f_c, of eq. 22a, to a maximum of 740 lb., for $n = 15$ and $f_c' = 2000$, would insure against rather high "design" concrete stresses for the higher steel ratios.

(Referring to Mr. Sutherland's letter, p. 826). The moment resisting bending (buckling) of a reinforced concrete column shaft is the resisting moment of a double-reinforced beam, it is not determined as any function of any radius of gyration, like the resisting moment of a homogeneous beam, equation c. The radius of gyration of a reinforced concrete column—properly stated: The R of its cross-sectional area—is not different from that of any other column of same cross-sectional area and shape. To assign a different R to columns of different materials, though of same cross-sectional area (and shape), is merely a transference of the difference in strength to a fictitious R. "Tentative Regulations", Sec. 1108, gives the formulae 26 and 26a, for long columns, as functions of such a fictitious R, the latter *to be computed* as directed in 1108c. The equations i and j, given by the writer, simply "fill the prescription" laid down in 1108c, *and no more.* They do not "define the writer's conception" of the radius of gyration of a reinforced concrete column. How would the correspondent fill this prescription or, going back to the source, how would the author of "Tentative Regulations" want to see his prescription filled?

The correspondent reopens the discussion of the writer's 1927 paper,*quoting six lines from p. 126-7, and says "the obvious impossibility of this analysis has already been pointed out by others." He does not mention the writer's reply to the "others," on p. 424, 1928 *Proceedings.* Whether they are "of the same opinion still" the writer does not know. Inasmuch as the correspondent says "the well-known formula relating external load on the column to internal fibre stresses *is based on the*

assumption of the lack of internal stress in the materials" the writer wishes to emphasize: equations 1, 2 and 3, p. 126, are *valid* and *correct, with* the internal stresses f_{cr} and f_{si} *present* in the column *prior* to the application of any external load P_e just the same as if there were *no* internal stresses in the materials, provided *only that* $\frac{Pe}{A} \geqq \frac{E}{n}\, m\,(1-p)$. [See p. 128-131.] The "design" loads $\frac{Pe}{A}$ for the several steel ratios p (with the possible exception of the lower steel ratios up to $p = .015$, m taken as .0005) being greater than the respective internal loads $\frac{E}{n}\, m\,(1-p)$, it is not necessary to "assume the lack of internal stresses in the materials." There are *no* internal stresses in the column *prior* to the action of the "internal" load, and equations 1, 2 and 3, give the relation between load and concrete and steel stresses just as correctly for this "internal" load as for any external load P_e by simply substituting $P_i = \frac{E}{n}\, m\,(1-p)$ for P in the said equations, and there is no minimum requirement for this load P_i. The "internal" load produces the compr. stresses of the concrete and the steel, f_{ci} and f_{si} of equations 6 and 7, respectively. Remembering that the tensile stress f_h (equation 4) of the concrete, developed when its natural contraction meets with resistance, is the cause of this load, it is evident that this load Pi applied to the column reduces the initial tensile stress f_h by the amount of the compr. stress f_{ci} due to this load, and the (smaller) tensile stress f_{cr} (the "residual tension") is the resultant:

$$f_{cr} = f_h + f_{ci} = -\frac{p}{1 + (n-1)\,p} \times Em \quad \text{(eq. 8)}$$

The equilibrium of the (total) concrete and steel stress in the column, viz. $f_{cr}\,(1-p) + f_{si} \times p = 0$ is satisfied by the values of f_{cr} (eq. 8) and f_{si} (eq. 7). The ratio of these stresses is $\frac{f_{cr}}{f_{si}} = \frac{p}{1-p}$ (see table p. 152). *On the other hand,* the mathematical terms of the stresses f_{cr} and f_{si}, when written as *functions of the resp. moduli and deformations,* are equations b′ and c′ on p. 157. They are identical with equations 8 and 7, respectively. Equations b′ and c′ prove the correctness of the former method of evolving these (and other) stresses, on p. 127.

The mathematical terms for the several stresses f_{oi}, f_{si} and f_{cr} *have* a physical meaning, and *no new* evidence is required to sustain the writer's criticism on p. 140-2.

Floor Test in the George Mason Hotel, Alexandria, Virginia

BY WILLIS A. SLATER*

THE BUILDING

The George Mason Hotel at Alexandria, Virginia is a six-story building approximately 113 ft. long and 100 ft. wide. Fig. 1 shows the general plan at the first floor level, the overall dimensions and the location of the three panels to which a test load was applied. The building has reinforced concrete columns with concrete beams extending from column to column which divide the floor space into rectangular panels. The floor slab of each panel is supported by the four beams which extend from column to column and without the use of intermediate beams.

The exterior walls are of brick. The exterior edges of the first floor slab rest upon the basement wall. The author was informed that this wall was built after the slab was constructed. The exterior edges of the slabs above the first floor are carried by reinforced concrete beams extending between wall columns.

The slabs are of reinforced concrete poured integrally with the beams, and are reinforced in two directions with the bars extending parallel to the sides of the panels. The slab for each panel is of uniform thickness throughout the panel. For the slabs of the first two floors the tension reinforcement in any panel has the same sectional area in either direction for negative as for positive moment, except that the sectional area of the reinforcement for negative moment at the exterior walls is only half as great as that for positive moment in the center of the panel.

The arrangement of the tension reinforcement for the beams is similar to that of the slabs; that is, the negative and the positive reinforcement are equal in sectional area except at wall

*Director Fritz Engineering Laboratory, Lehigh University.

columns where the area of the tension reinforcement is only half as great as that for positive moment at the center of the span.

The above statements regarding the amounts of reinforcement, for the positive and negative moments apply to the panels tested. Since one-half the negative reinforcement at each interior support consists of bars carried through from the adjoining panel, the areas of the positive and the negative reinforcement are slightly different where the size or spacing of bars in one panel differs from that in the adjoining panel.

GENERAL DESCRIPTION OF TEST

A portion of the first floor consisting of three panels as shown in Fig. 1 at the southeast corner of the building, was tested.

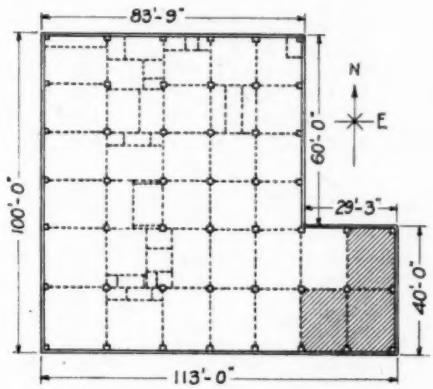
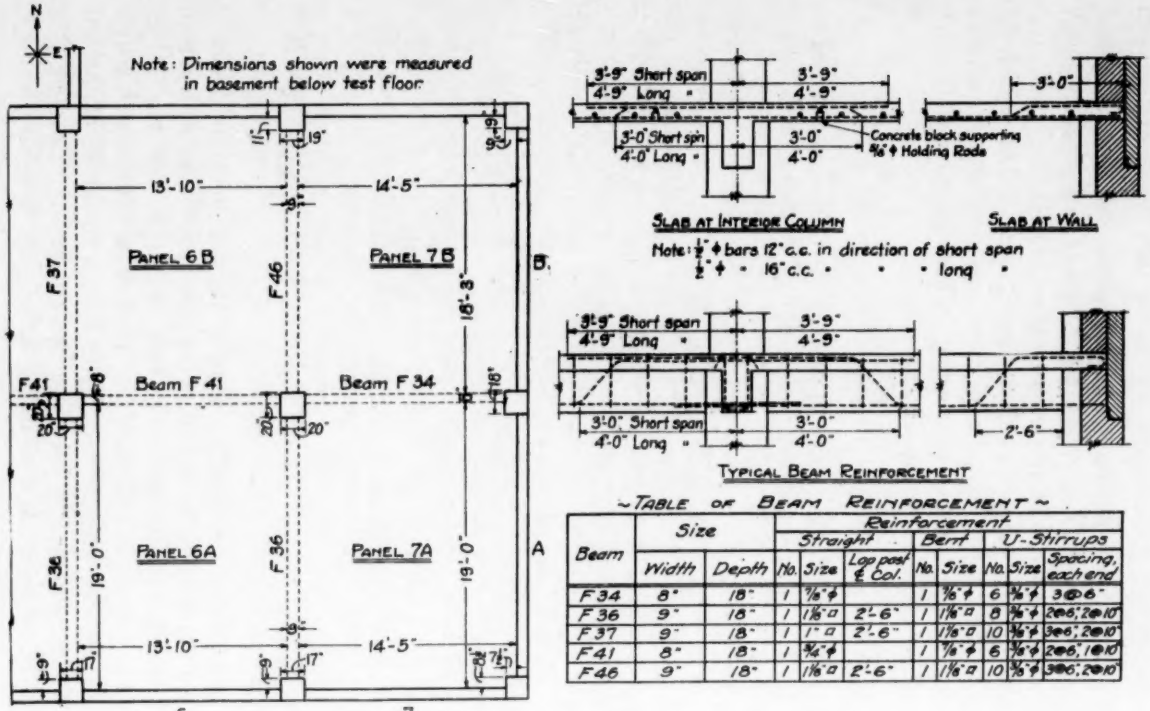


FIG. 1—SKETCH OF FIRST FLOOR PLAN SHOWING PANELS LOADED.

The dimensions and reinforcement of the test floor are shown in Fig. 2. Fig. 3 gives a view of the under side of the slab during the test. The design load is 143 lb. per sq. ft. Of this 75, 56, and 12 lb. per sq. ft. represent live load, weight of slab and weight of wood flooring respectively. The design thickness of the slab was 4½ in. for each of the three panels tested. The measured thickness varied from 6.05 in. to 4.52 in. as shown in Table 1. The distance from the compression surface to the center of the tension reinforcement varied from 5.06 to 2.24 in. as shown in



~TABLE OF BEAM REINFORCEMENT~

Beam	Size		Reinforcement							
			Straight			Bent		U-Stirrups		
	Width	Depth	No.	Size	Lap past ℄ Col.	No.	Size	No.	Size	Spacing, each end
F 34	8"	18"	1	⅞" ϕ		1	⅞" ϕ	6	⅜" ϕ	3@6"
F 36	9"	18"	1	1⅛" □	2'-6"	1	1⅛" □	8	⅜" ϕ	2@6", 2@10"
F 37	9"	18"	1	1" □	2'-6"	1	1⅛" □	10	⅜" ϕ	3@6", 2@10"
F 41	8"	18"	1	¾" ϕ		1	⅞" ϕ	6	⅜" ϕ	2@6", 1@10"
F 46	9"	18"	1	1⅛" □	2'-6"	1	1⅛" □	10	⅜" ϕ	3@6", 2@10"

FIG. 2—DIMENSIONS AND DESIGN OF TEST FLOOR.

TABLE I

MEASURED THICKNESS OF SLABS, DEPTH TO BARS AND DEPTH FROM THE COMPRESSION SURFACE TO CENTER OF REINFORCEMENT FOR EACH GAGE LINE.

Location	Gage Line No.	Total Thickness of Slab	Thickness of Concrete over Bars	Depth from Compression Surface to Center of Bars
		Inches	Inches	Inches
Over beam at N. side of panel 6A	6	4.83	1.67	2.91
	7	4.95	1.25	3.45
	8	4.55	1.87	2.43
	9	5.26	1.80	3.21
	Average		1.65	3.00
Over beam at N. side of panel 7A	14	4.67	1.15	3.27
	15	4.60	1.56	2.79
	16	4.53	1.97	2.31
	Average		1.56	2.79
Over beam at W. side of panel 7A	19	4.73	2.17	2.31
	20	5.40	2.49	2.66
	21	5.44	2.48	2.71
	22	5.17	2.68	2.24
	Average		2.45	2.48
Panel 7A at S. wall	24	4.52	0.86	3.41
	25	4.74	1.24	3.25
	Average		1.05	3.33
Panel 7A at E. wall	26	4.70	1.00	3.45
	27	4.90	1.38	3.27
	28	4.74	1.84	2.65
	Average		1.40	3.12
Panel 6A at S. wall	34	4.69	0.83	3.61
	35	5.04	1.13	3.66
	Average		0.98	3.63
Panel 6A along E. & W. center line	40	5.59	1.59	3.75
	41	5.85	1.10	4.50
	42	5.90	1.41	4.24
	Average		1.37	4.16
Panel 7A along E. & W. center line	47	5.97	1.40	4.32
	48	5.81	1.66	3.90
	49	5.44	1.84	3.35
	Average		1.63	3.86
Panel 7A along N. & S. center line	52	5.15	1.00	3.90
	53	5.34	0.80	4.29
	54	5.76	1.04	4.47
	55	5.64	0.44	4.95
	56	5.09	1.04	3.80
	Average		0.86	4.28
Panel 6A along N. & S. center line	63	5.23	1.06	3.92
	64	5.59	0.84	4.50
	65	6.05	0.74	5.06
	66	5.29	0.97	4.07
	67	5.42	1.36	3.81
	Average		0.99	4.27

Table 1. At the time of the test the slab was nearly four months old. The concrete was hard and appeared to be of good quality. Three tests made on two ½-in. bars taken from the slab in nearby panels gave an average yield point stress of 36,900 lb. per sq. in. based upon the sectional areas determined from weights of known lengths of bar. The areas so determined were about 4½ per cent less than the nominal areas.

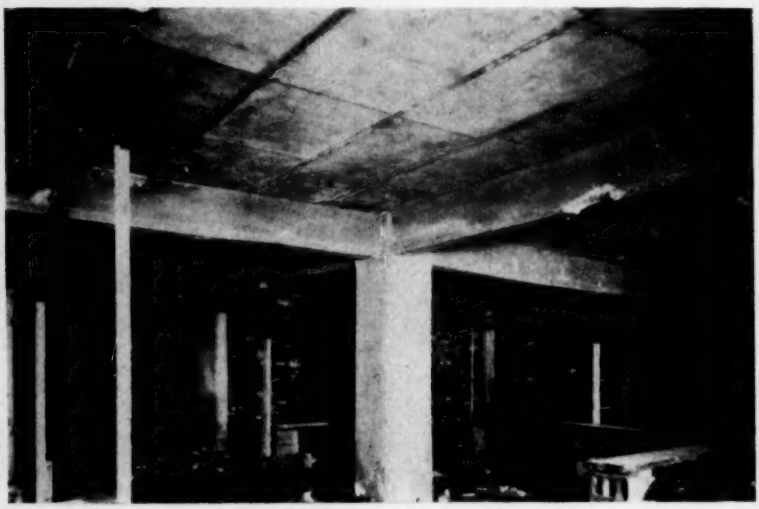


FIG. 3—VIEW OF UNDER SIDE OF SLAB DURING TEST.

The weather conditions at the time of the test were favorable for reliable strain gage data. During the loading and unloading the lowest temperature read on a thermometer whose bulb was embedded in plaster of paris in close contact with the bottom of the test slab was 70° F. and the highest was 78° F. About a week after the unloading of the slab the temperature had dropped to 62.5° and appears to have affected the deflection of the slabs.

LOADING

Second-hand common bricks were used as loading material. In order to determine the weight per cubic foot of the brick, weighings were made on two piles stacked as in loading the

floor. One pile was 5 ft. square and 22¼ in. (9 courses) high. The other pile was 2½ ft. square and 14⅞ in. (6 courses) high. The weights per square foot per course for the two determinations were 21.4 and 20 lb. respectively. In computing the test load the average, 20.7 was used. The load was applied in three stages. The intensities aimed at were 87, 162, and 230 lb per sq. ft. for these stages. Adding to these applied loads the design weight of the slab, 56 lb. per sq. ft. gives 143, 218, and 286 lb.

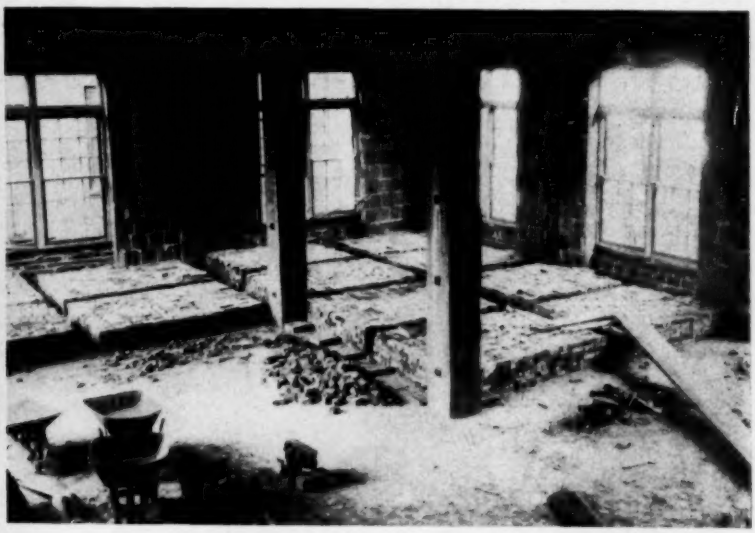


FIG. 4—VIEW OF TEST LOAD OF 87 LB. PER SQ. FT.

per sq. ft. as the total load sustained by the slab at the three stages. The first and third values are one and two times the total design load. The second value represents the dead load plus twice the live load. The highest load, 286 lb. per sq. ft. might also be stated as the total dead load plus 2.9 times the design live load. Table 2 gives the intensities of the loads actually applied and compares these with the values for the three different stages noted above. Figures 4, 5 and 6 are views of the test load at three stages.

In applying the load it was not possible to cover the entire area of the panel because it was necessary to leave aisle space to give access to the gage lines and to prevent arching of the bricks. Aisles were left on edges and center lines of panels. The location of loaded areas is shown in Fig. 7. The area loaded was 74 per cent of the total area of the test panels. The loads per square foot here reported were found by dividing the total panel loads by the total area of the panel.

Loading of the slab began on the morning of September 15, 1925. The first increment, of 87 (143 with weight of slab) lb. per sq. ft., was completed at noon. Strain and deflection read-

FIG. 5—VIEW OF TEST LOAD OF 162 LB. PER SQ. FT.

ings were taken and a search for cracks was made. Loading was resumed at 1:30 p. m. The load of 162 lb. per sq. ft. (applied load only) had been almost completed by evening of the same day and was entirely completed by 9:15 a. m. September 16. The loading was interrupted only long enough to permit the taking of observations, and the maximum applied load of 230 lb. per sq. ft. was in place by 3:00 p. m. September 16. This load was left in position for 24 hr. Removal of the load began on September 17 at 3:00 p. m. and was completed before 5:30 p. m. on the same day.

OBSERVATIONS

Strains were measured by means of a strain gage at 55 places on the reinforcement and at 9 places on the concrete. The location of the gage lines is shown in Figs. 7 and 8. The gage length used was 8 in. The methods used were similar to those used in other field tests by the writer. Deflections were observed at the centers of the test panels and the two adjoining unloaded panels, also at the centers of all beams supporting loaded panels except the north beam of panel 6A. The location of the deflection readings is shown in Fig. 8.

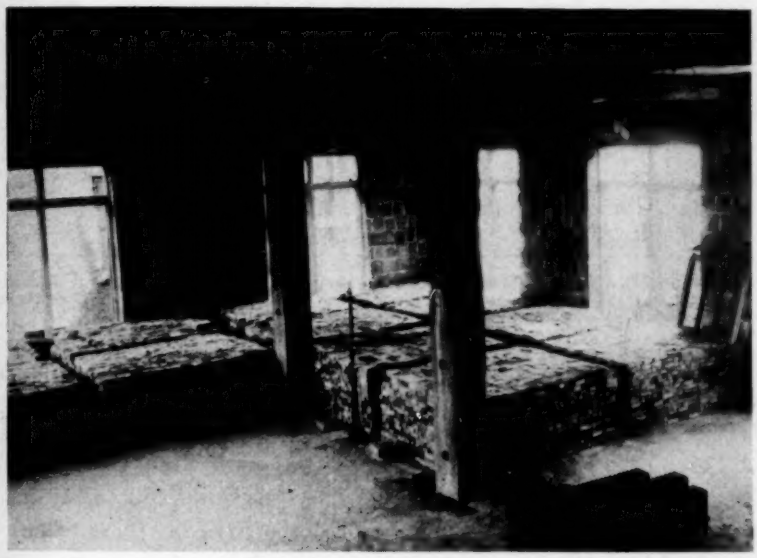


FIG. 6—VIEW OF TEST LOAD OF 230 LB. PER SQ. FT.

At a few typical positions the widths of the largest cracks were measured with a lens and scale. It is believed that the widths reported do not have an error greater than .005 in.

CRACKS

Previous to the test a search was made for cracks in the slab. Only one crack, a short one near column 16 in Fig. 7, was found

TABLE II

INTENSITY OF APPLIED LOAD AND COURSES OF BRICK IN PLACE AT DIFFERENT STAGES OF THE TEST

Stages of Loading	Panel	Ratio of Loaded to Total Area	Courses of Brick	Intensity of Load Lb. per Sq. Ft.		Ratio Test Load to Design Load	
				Over Area Loaded	Average	Total[1]	Live[2]
1	6A	.725	6	124	90	1.02	1.04
	7A	.709	6	124	88	1.01	1.01
	7B	.773	5½	114	88	1.01	1.01
		Average			89	1.01	1.02
2	6A	.725	11	228	165	1.55	2.04
	7A	.709	11	228	161	1.52	1.99
	7B	.773	10½	217	168	1.57	2.09
		Average			165	1.55	2.04
3	6A	.725	16	331	240	2.07	3.06
	7A	.709	16	331	234	2.03	2.96
	7B	.773	15	310	240	2.07	3.06
		Average			238	2.06	3.03

[1]Under Total loads the weight of the floor is included in both the test load and the design load.
[2]The *live* test load was taken as 12 lb. per sq. ft. less than the applied test load. This was in order to make allowance for the weight of the wooden floor which was not in place.

on the upper surface, but some of those found in the course of the test appear to have been old cracks freshly opened. The top of the slab was rough, and even after sweeping dust enough remained to make the finding cf fine cracks very difficult. The steel forms used for the slab left the bottom surface of the slab very smooth and even fine cracks were easily traced. The location of the cracks found on the under surface is shown in Fig. 8. From the circumferential course of many cracks about the corner columns of the building, it seems probable that they occurred prior to the construction of the brick walls under the edges of the floor slab. None of these cracks was large before the load was applied and they did not increase much in width during the loading. This is indicated by the measured widths given in Table 3. The location of the points where crack widths were measured is shown in Fig. 8.

Under the load of 87 lb. per sq. ft. plus the weight of the slab (total design load) a crack was found on the upper surface of the slab along the east edge of the beam which lies between panels 6A and 7A. Another was found along the north edge of the beam between panels 7A and 7B. These cracks were both about .01 in. wide. Although they had not been found previous to applying the load they had the appearance of old cracks. They were probably old cracks which were so nearly closed that the roughness of the floor and the presence of dust were sufficient to prevent their detection previous to the loading. At this load no cracks were found on the top of the slab close to the walls though careful examination was made there. The brick and tile wall gave restraint at the exterior edges of the slab, and, if the wall had been present when the cracks along the interior beams formed, cracks probably would have formed along the wall supports also for some of the measured stresses at the wall under the maximum load were very high. It seems likely, therefore, that the cracks along the interior beams formed before the exterior walls were built. An examination made after the load of 162 lb.

TABLE III

MEASURED CRACK WIDTHS

Crack Designation (Fig. 8)	Crack Widths at Applied Loads of		
	0	89 Lb. per Sq. Ft.	238 Lb. per Sq. Ft.
	Inches	Inches	Inches
		On Under Surface of Slab	
A	0.015	0.015	0.020
B	0.012	0.010	0.005
C	0.005	0.007	0.015
D	0.005	0.002	0.005
E	0.010	0.015	0.015
F	0.005	0.005	0.010
G	0.010	0.004	0.010
H	0.010	0.005	0.010
I	0.002	0.001	0.001
K		0.020	0.015
		On Upper Surface of Slab	
L			0.020
M			0.005
N			0.020
O			0.040

per sq. ft. was in place disclosed no new cracks on the top of the slab. At the maximum load cracks had formed along the walls in all the loaded panels and along the edges of all beams which supported the loaded slabs. They were present on both edges of both the beams which were between loaded panels. New cracks had formed also on the under surface of the slab. The location of the cracks on the upper surface of the slab is shown in Fig. 7

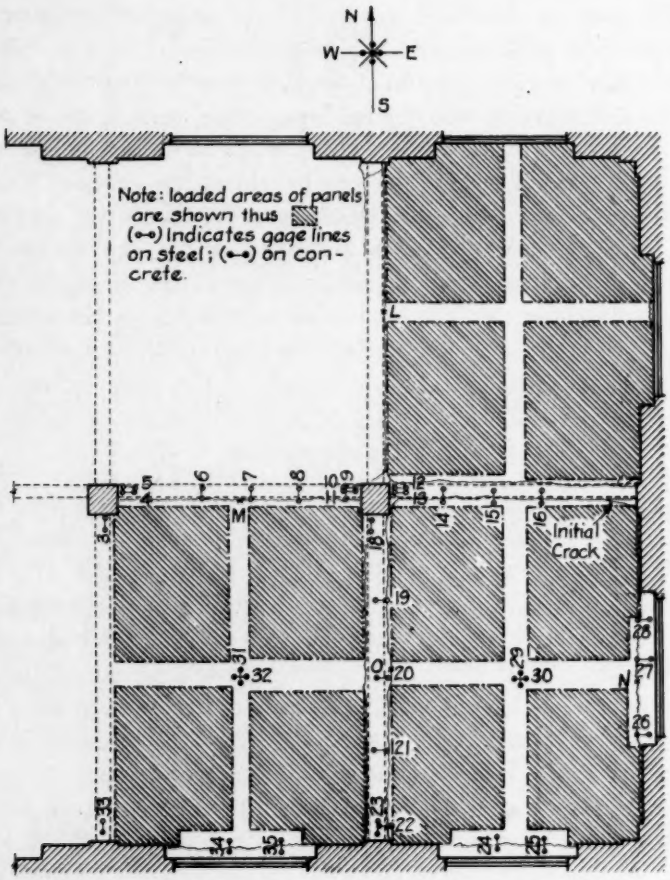


FIG. 7—LOCATION OF LOAD AREAS, AISLES, GAGE LINES AND CRACKS ON TOP OF SLAB.

and the location of those on the under surface is shown in Fig. 8. The measurement of crack widths at certain places was attempted. On account of the roughness of the top of the slab these measurements are less reliable than those for crack widths on the bottom of the slab.

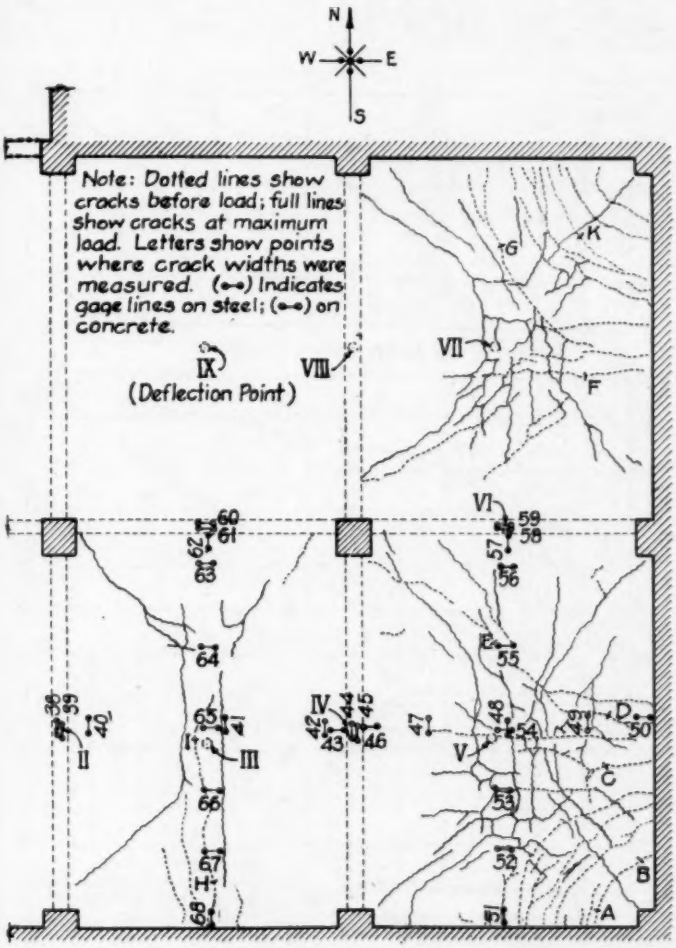


FIG. 8—LOCATION OF GAGE LINES, DEFLECTION POINTS AND CRACKS ON UNDER SIDE OF SLAB.

DEFLECTIONS

The deflections are shown in Figs. 9 and 10. The maximum deflection occurred at the center of panel 7A after the maximum load had been in place 24 hr. and amounted to .58 in. Considering the span for this wall panel as the distance from the center of the beam on one side to a point 3 in. within the supporting wall on the other side, the deflection was 1/311 of the span. The deflection

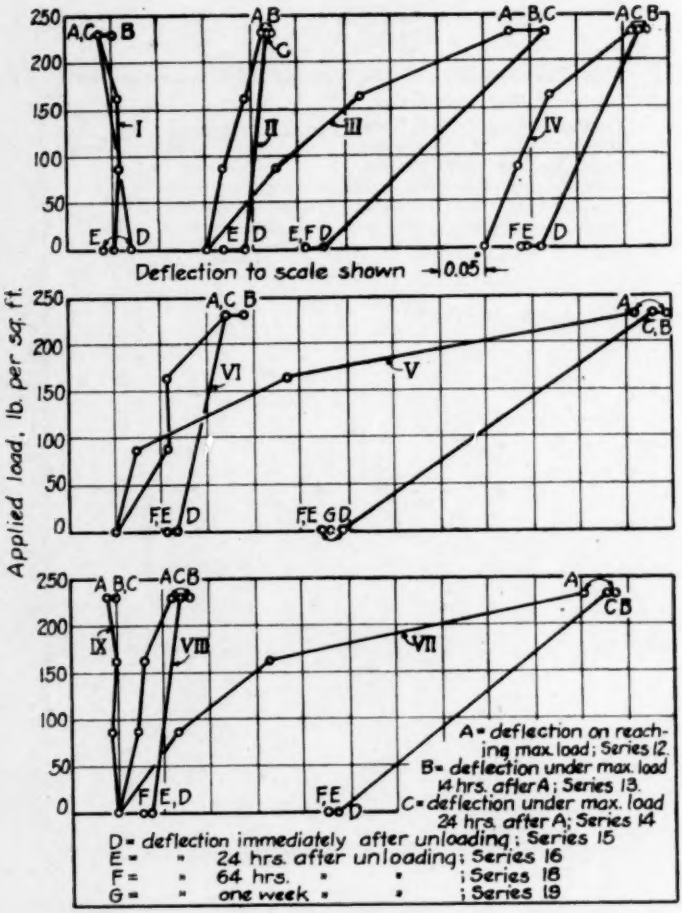


FIG. 9—LOAD-DEFLECTION CURVES FOR SLABS AND BEAMS.

in panel 7B was slightly less, and that in panel 6A was considerably less than that in panel 7A.

It will be seen that the unloaded panels adjoining the loaded panels had only a very slight upward deflection. The beam between the loaded and unloaded panels, however, had some downward deflection and if the deflection of the slab be corrected for the deflection of the girders the maximum slab deflection is found to be about 0.51 in. In the same way the maximum upward deflection of the unloaded slab adjacent to panel 6A is found to be

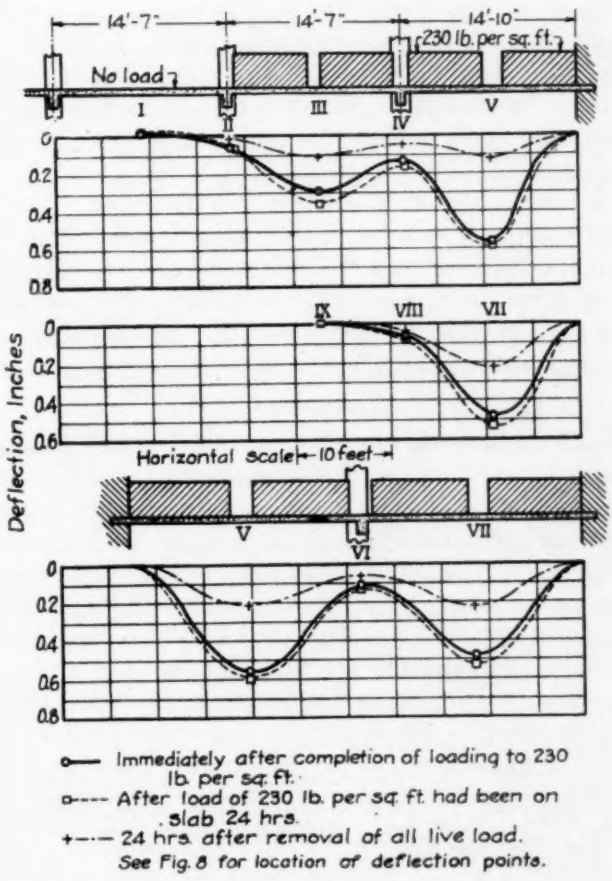


FIG. 10—PROFILES OF DEFLECTED FLOOR.

about .04 in. Twenty-four hr. after the load had been removed from the slab the deflection was less than it was after it had been off for a week. The fact that the unloaded as well as the loaded slabs were deflected more at a week than at one day after removal of the load makes it appear that the cause was a change in weather conditions. While it has been found in other cases* that a temperature change has affected the deflection it is not easy by analysis to see what the nature of the change should be. The temperature was about 10 or 12 degrees higher at the time of removal of the load than it was a week later.

As shown in Fig. 9 the deflection at the center of panel 7A decreased from the maximum value of 0.58 in. under full load to 0.24 in. immediately after removal of the load and to 0.21 in. 24 hr. later. A week later the deflection had increased to 0.23 in. This probably was a result of the temperature change previously mentioned.

The recovery, that is, the decrease in deflection, was a maximum for most cases 24 hr. after the load had been removed. The maximum deflections and the recoveries for all three loaded panels and for the beams between loaded panels are given in Table 4. The average recovery was 66 percent, the maximum was 74 percent and the minimum was 59 percent.

STRESSES

The load-stress curves have been plotted in Figs. 11 and 12 and the stresses on the sections for which moment coefficients were computed are given in Figs. 13, 14, and 15. In computing the stresses from the measured strains the modulus of elasticity of the steel was assumed as 30,000,000 lb. per sq. in. The rate of increase of stress with increase of load was greater for the high loads than for the low loads. This phenomenon has been observed in other tests and it is likely that it is due to the formation of cracks throwing a larger proportion of the total stress to the steel at the higher than at the lower loads. Under the discussion of cracks reasons were given for believing that the cracks over the two beams between loaded panels were present before the test started but that those at the walls were not. Added reason for this belief is found in the difference in the rates of increase of stress at the two

*Schulze Baking Company Building, Bulletin 84, University of Illinois Engineering Experiment Station, p. 78.

TABLE IV

FULL LOAD DEFLECTIONS, RESIDUAL DEFLECTIONS AND RECOVERIES FOR LOADED PANELS

Panel or Beam	Center Deflections			Recovery
	Under Maximum Load 24 Hours	Under Zero Load (Residual)		
	Inches	Inches	Inches	Per Cent
6A	0.366	0.106	0.260	71
7A	0.580	0.210	0.370	64
7B	0.526	0.216	0.310	59
Beam between 6A and 7A panels	0.170	0.044	0.126	74
Beam between 7A and 7B panels	0.140	0.054	0.086	61
Average				66

places. Reference to Fig. 11 shows that in gage lines 26, 27, and 28 the increase of stress at the early stages of the test was very slow as though the concrete was taking nearly all the stress. In gage lines 19, 20, 21, and 22 the rate of increase in stress was much more rapid at the beginning of the test as though the concrete was already cracked. For these latter gage lines there is, however, a very rapid increase of stress at a load of about 180 lb. per sq. ft., and this requires explanation if it is not due to the formation of cracks across these gage lines. It has been noted that, on the under side of the slab in the interior of the panel and on the upper side near the wall, cracks were not found at the second stage of the loading, that is at 162 lb. per sq. ft. but that they were very evident at the maximum load. This accounts directly for the rapid increase of stress in gage lines 53 and 54 at about 180 lb. per sq. ft. The yielding at this place would throw a larger proportion of the total moment to the support and this probably explains the rapid increase in the stresses at gage lines 19 to 22 for a load of about 180 lb. per sq. ft. The load stress curves of Fig. 11 indicate that for gage lines 19 to 22 a test load

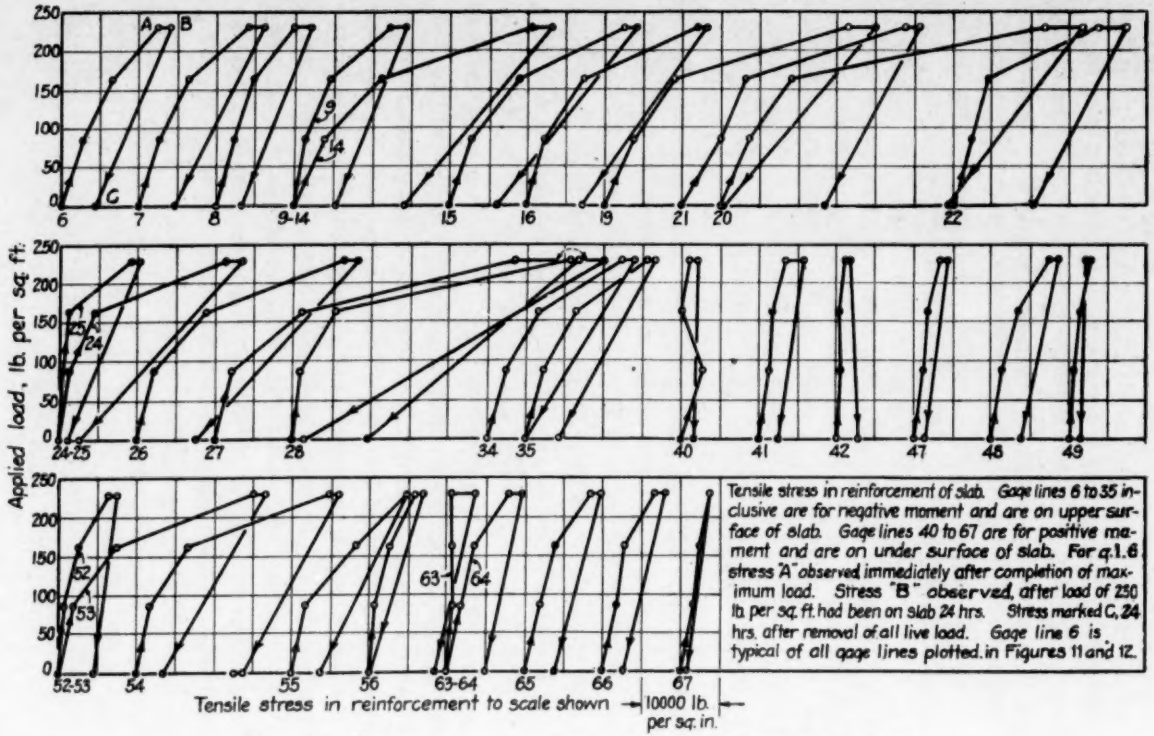

Applied load, lb. per sq. ft.
Tensile stress in reinforcement to scale shown
10000 lb. per sq. in.
Tensile stress in reinforcement of slab. Gage lines 6 to 35 inclusive are for negative moment and are on upper surface of slab. Gage lines 40 to 67 are for positive moment and are on under surface of slab. For gage lines 6 stress "A" observed immediately after completion of maximum load. Stress "B" observed, after load of 250 lb. per sq. ft. had been on slab 24 hrs. Stress marked C, 24 hrs. after removal of all live load. Gage line 6 is typical of all gage lines plotted in Figures 11 and 12.

FIG. 11—LOAD-STRESS CURVES FOR SLAB REINFORCEMENT.

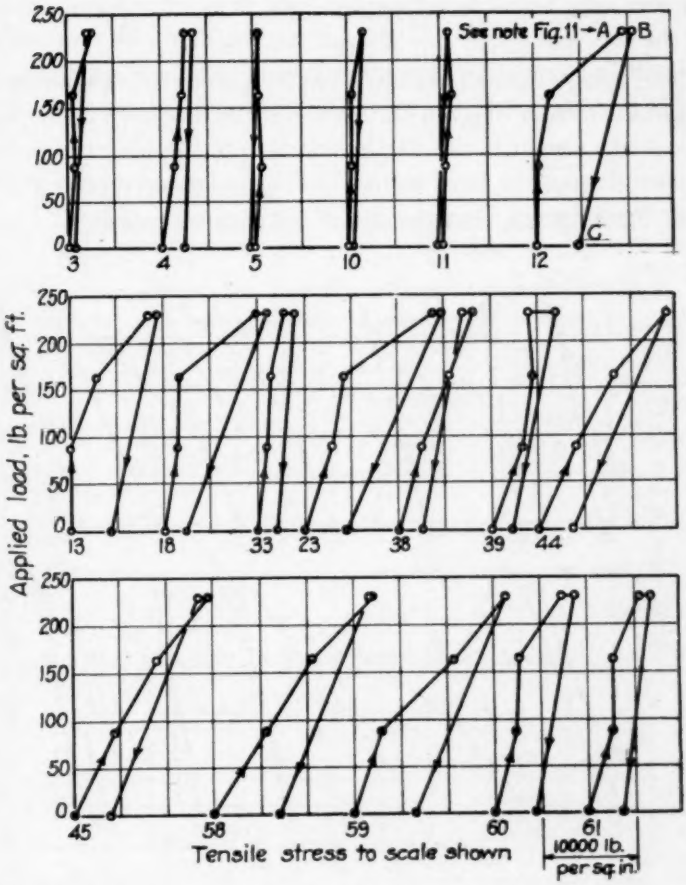


FIG. 12—LOAD-STRESS CURVES FOR BEAM REINFORCEMENT.

equal to the weight of the slab caused a stress in the reinforcement of 1500 to 2000 lb. per sq. in. The stress at this place before the test load was applied must have been as much as this. At other places, for example, gage lines 26, 27, and 28, the initial slope of the load stress curves indicates an initial stress very much smaller. Yielding of the green concrete under load may, however, have been sufficient to permit the reinforcement to take

stresses as great as those at gage lines 19 to 22 without cracking of the concrete.

The stress at gage lines 20, 27, and 28 exceeded the yield point determined from the coupon test and if the unknown initial stress due to the weight of the slab were added to the observed stress it is possible that the yield point would have been exceeded in other gage lines also.

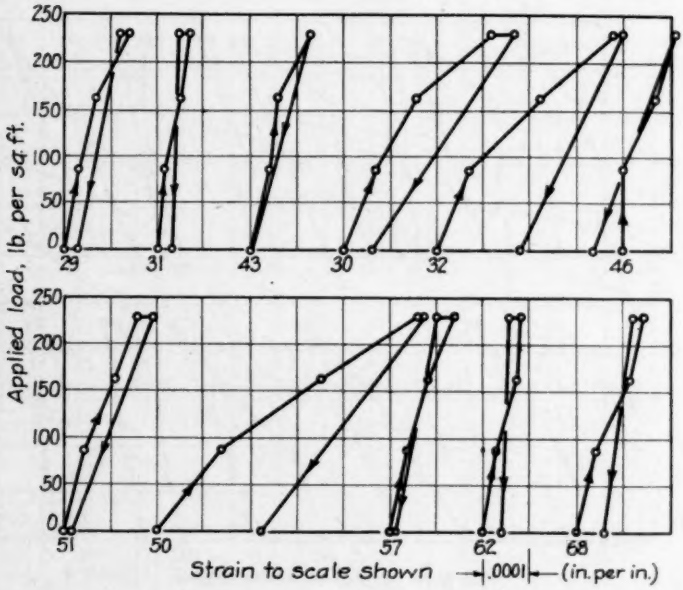


FIG. 13—LOAD-STRAIN CURVES FOR CONCRETE.

In Fig. 11 it will be seen that on the removal of the load there was in nearly all cases a residual stress or at least a residual strain. In most cases a part of the residual stress (or strain) was likely due to the weight of the slab. In some cases it was probably due to the yield point of the bars having been exceeded. In other cases it may have been due to dirt getting into the cracks and preventing their closing. The latter cause seems to have been minor since the residual stress appears to have been as great in

the bars on the bottom of the slab where dirt could not easily get into the cracks as it was on the top.

The modulus of elasticity of the concrete is not known, therefore the stresses in the concrete are not known. The largest

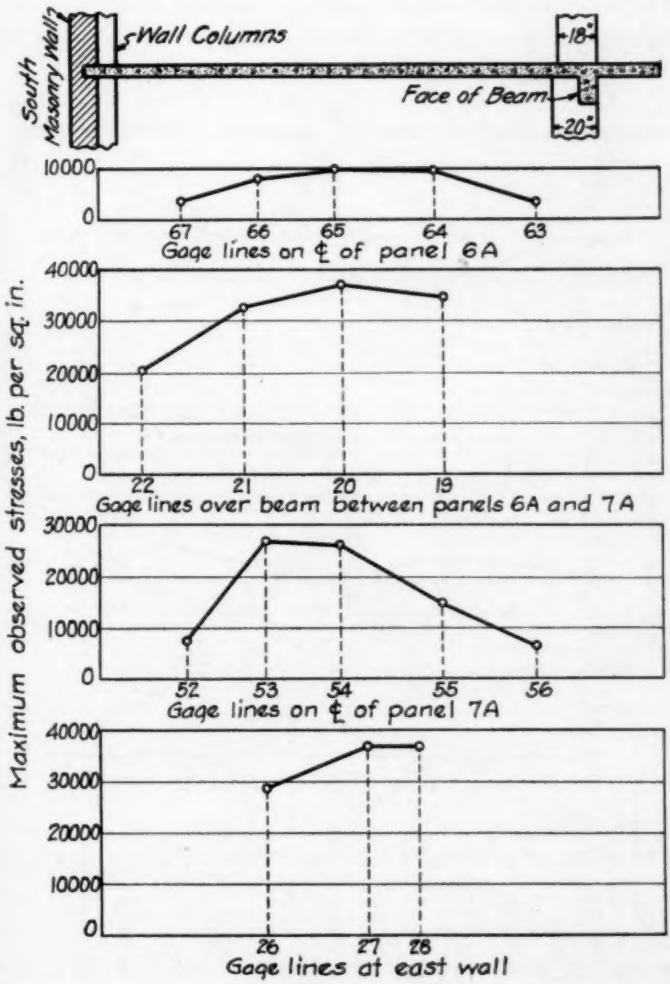


FIG. 14—STRESSES ON SECTIONS FOR WHICH MOMENT COEFFICIENTS WERE COMPUTED; SECTIONS CUTTING BARS WHICH EXTEND IN DIRECTION OF SHORT SPAN.

strain measured however, was 0.00057 at the maximum load and it is likely that the stress did not exceed 55 or 60 percent of the ultimate strength of the concrete.

MOMENTS IN SLABS

The resisting moment, M, across a section of the slab may be computed as $M = Af_s jd$ where A is the sectional area of all the

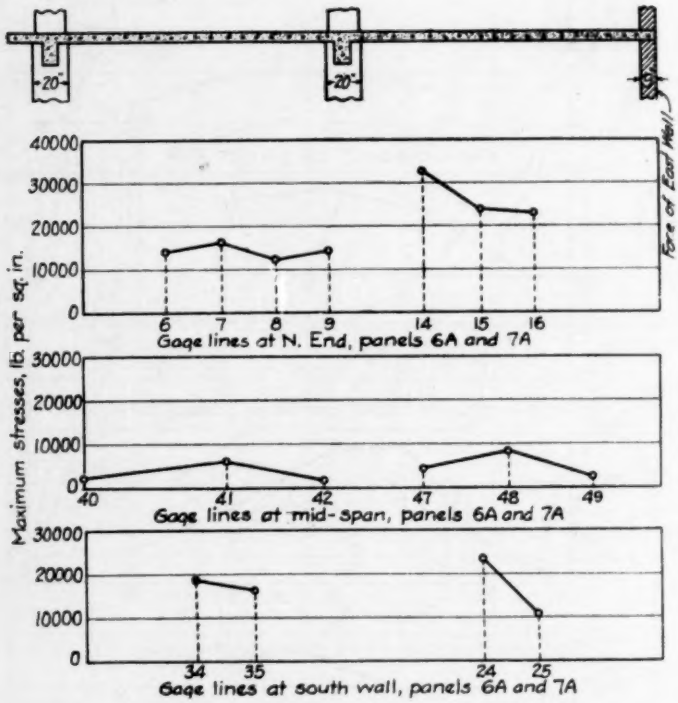


FIG. 15—STRESSES ON SECTIONS FOR WHICH MOMENT COEFFICIENTS WERE COMPUTED; SECTIONS CUTTING BARS WHICH EXTEND IN DIRECTION OF LONG SPAN.

bars crossing the section, f_s is the observed stress in the reinforcement and jd is the distance from the center of the reinforcement to the resultant compressive force. Putting this moment equal to KW1 the value of the coefficient K may be computed. The

TABLE V.

MOMENT COEFFICIENTS FOR THE SLABS AND BEAMS

A. Moment Coefficients for the Slabs

Panel	Positive or Negative	Location	Direction (Short or Long Span)	Gage Lines	Average Depth in In.	Stresses Lb. per Sq. In.		Moment Coefficients Based on	
						Maximum	Average	Maximum Stress	Average Stress
7A	Negative	East wall	Short	26 to 28	3.12	37000	34200	0.0148	0.0137
7A	Negative	First interior support	Short	19 to 22	2.48	37000	31200	0.0236	0.0199
7A	Negative	South wall	Long	24 & 25	3.33	23600	17000	0.0058	0.0042
7A	Negative	First interior support	Long	14 to 16	2.79	33000	26700	0.0136	0.0110
7A	Positive	Center of panel	Long	47 to 49	3.86	8500	4900	0.0049	0.0028
7A	Positive	Center of panel	Short	52 to 56	4.28	26800	16500	0.0294	0.0181
6A	Negative	South wall	Long	34 & 35	3.63	18900	17600	0.0053	0.0049
6A	Negative	First interior support	Long	6 to 9	3.00	16300	14300	0.0075	0.0066
6A	Positive	Center of panel	Long	40 to 42	4.16	5700	3000	0.0037	0.0019
6A	Positive	Center of panel	Short	63 to 67	4.27	9800	7000	0.0114	0.0081

B. Moment Coefficients for the Beams

Beam Designation	Positive or Negative	Location	Gage Lines	Average Depth in In.	Stresses Lb. per Sq. In.		Average Moment Coefficients	
					Maximum	Average	For Position of Gage Lines	For Face of Column
F34	Negative	Interior column	12 & 13	15.87	10500	9850	0.0341	0.0441
F34	Positive	Mid span	58 & 59	15.70	17000	16550	0.0568	
F36	Negative	Interior column	18	13.92	11100	11100	0.0338	0.0406
F36	Negative	Wall column	23	14.39	15000	15000	0.0235	0.0291
F36	Positive	Mid span	44 & 45	16.24	14100	13900	0.0492	

coefficients so computed are given in Table 5. The load, W, used in the computations is the sum of the dead load and the maximum live load in the entire panel. The span is the clear span between edges of the supports whether of beam or wall. Two different stresses were used, (a) the average for all the gage lines within the section under consideration, and (b) the greatest stress observed in the section. The coefficients based on average stresses are useful for comparing the total moments on the different sections. Those based upon the greatest observed stress within the section indicate the coefficient which should be used in design in order that no stress should be greater than the assumed working stress. In two places the observed stress had exceeded the yield point and in those cases the yield point stress (37,000 lb. per sq. in.) was used for computing the coefficients instead of the stress indicated by multiplying the strain by the modulus of elasticity.

In computing the coefficients some allowance should be made for initial stress in the reinforcement due to the weight of the slab. This was not done because the amount of the initial stress was too uncertain. It does not seem likely, however, that the initial stress exceeded 2000 lb. per sq. in.

In the direction of the short span of panel 7A the negative resisting moment of the observed stresses at the interior support was 1.10 times the positive moment of the observed stresses at the center of the span. That the ratio of negative to positive moment was not greater probably is due to the fact that the restraint at the wall was less than that at the interior support. Another and possibly more important factor is the fact that the construction provided for a negative moment at the interior support only 0.58 as great as that for the positive moment. The fact that a larger proportion of the total moment was resisted as negative moment than that which the design provided for, shows that a still larger proportion should have been provided as negative moment. It was probably intended in the design that the positive and the negative moments should be equal as the steel areas are the same for these places. In the construction, however, the effective depth to the reinforcement for negative was only 0.58 as great as that for the positive moment.

In a similar manner the resisting moment provided at the wall support of the short span was only 0.36 while that developed by

the stresses in the test was 0.76 of that for positive moment in the same direction. This indicates that in the design of a slab built into a brick wall such as that used here the negative moment at the wall should be taken as greater than 0.76 of the positive moment. How much greater it should be, the data do not show, but it is likely that the negative moment provided at the wall support (where the slab is supported by a solid wall and carries a wall of one story height or more) should not be less than the positive moment in the same panel.

Comparisons of the moments at the wall support, the center of the span and the interior support for the long span of the same panel, 7A, result in conclusions of the same nature as those discussed for the short span.

The moment coefficients for the slabs were in general somewhat less than those found in the test reported in Technologic Paper 220 of the Bureau of Standards. The stresses from which these coefficients were computed range from 5700 lb. per sq. in. to the yield point, 37,000 lb. per sq. in. The stresses used in Technologic Paper No. 220 ranged from 30,000 to 50,000 lb. per sq. in. In such tests the moment coefficients have generally been found to increase somewhat as the stresses on which they are based increase. The difference in stresses in these two tests is probably sufficient to account for the difference in moment coefficients. In view of this difference in stresses it may be concluded that the data of this test are in reasonable accord with those reported in Technologic Paper 220 except that the ratio of negative to positive moment is entirely different for the two tests. The difference in effective depth at positions of positive and of negative moment would lead one to expect a larger ratio of positive to negative moment here than that found in the former test.

The difference in effective depth at the positions of positive and negative moment apparently comes from two sources, the greater thickness of slab at the middle of the panel and the greater depth of embedment of the bars at the support. The bars were not too close to the bottom of the slab in the middle of the panel but they could with advantage have been held closer to the top at positions of negative moment, especially along the long side of the panel where the importance of adequate resistance for negative moment is greater than along the end of the panel.

MOMENTS IN BEAMS

The moment coefficients, K, $\left(= \frac{Af_s jd}{w_1}\right)$ for the two beams which were fully loaded were computed in a manner similar to that used for the slabs. For determining the load, W, the beam between panels 7A and 7B was assumed to carry the load from an area bounded by intersecting lines drawn from the ends of the beam at 45 degrees with the direction of the beam. The beam between panels 6A and 7A was assumed to carry the load from an area bounded by the longitudinal center lines of the panels and lines at 45 degrees with the beam intersecting at the ends of the beam. For the purpose of computing the loads the wall end of either beam is taken at the face of the wall, and the column end is taken as at the intersection of the center lines of the beams. The coefficients computed in this way are given in Table 5. Those marked "for position of gage line" are computed from the stress f_s actually observed. Those marked "for face of column" were computed for a corrected stress which was assumed to exist at the face of the column. The stress at the face of the column was estimated by assuming that the stress was directly proportional to the distance from the point of inflection toward the column. For this purpose the point of inflection was assumed to be one-fifth of the clear span from the face of the column.

It will be seen in Table 5 that the positive moment was greater than the negative moment for both beams. Several factors probably contribute to this result. First, unbalanced moments were applied to the columns at both ends of both beams and the bending of the columns must have permitted a decrease in the negative moment and an increase in the positive moment. Second, most likely the slab bars for some distance on either side of the beam assisted in resisting the negative moment, but in computing the coefficients shown in Table 5 only the bars within the width of the beam were assumed to participate in carrying the moment in the beams. Therefore, the negative moments probably were greater than the coefficients indicate them to be. Third, it has repeatedly been found that, the higher the observed stresses, the higher the proportion of the applied moment accounted for by the stress in the reinforcement. Since the highest observed stress in either of the beams was only 16,550 lb. per sq.

in. it is likely that with higher stresses larger moment coefficients would have been found.

The positive moments found from the stresses were greater than 1/24 W1, the moment at the center of the span of a fixed beam uniformly loaded. The influences mentioned above as causing the moments computed from the observed stresses to be small affect principally the negative moment and it is possible that the positive moments were not far from the true moments.

INSPECTION OF FLOOR AFTER FOUR YEARS' SERVICE

The test described in the preceding pages was made in September, 1925. In October, 1929, after the hotel had been in service approximately four years, the authors made an examination of the floor to which the test load had been applied. A dining room occupies the area of the test panels. Since the test, the concrete floor has been covered with a tile surface made up of white glazed tiles about 1 in. square with thin mortar joints extending parallel to the panel edges. The first impression was that there were no cracks in the tile surface, but closer examination revealed a crack estimated to be about .01 in. wide along each beam examined, both within and outside of the area which had been occupied by the test load. The crack over the beam between bays 6 and 7 (Fig. 2) lay approximately over the east edge of the beam. The crack over the next beam farther west was over the west edge and that over the third beam was over the east edge. The cracks extending east and west lay for two panel lengths over the north edge of the beams extending east and west between bays A and B, Fig. 2. In the third panel length it lay over the south edge of the beam. No cracks were found along the panel edges at the walls of the building.

The location of the cracks along lines where high bending moments existed in the test associates the cracks with flexural phenomena, and the fact that no cracks were found along the walls may probably be explained by the fact that the furring and plaster on the wall extended out so far as to cover the section of maximum moment and hide existing cracks from view. However, it is not unlikely that contraction with temperature changes played an important part in causing the cracks and that the sections of maximum flexure merely served to localize them.

More important probably than the presence of the cracks is the fact that they were so small as to escape detection except on close examination. They were much less apparent than the cracks frequently found in terrazzo surfaces on what are presumably conservatively designed floor structures.

On the under surface of the test slab, the pencil marks were still present which marked the cracks as they developed with the test load. There was no indication that any of these cracks was larger than just after the removal of the test load.

In this brief inspection no measurements could be taken to detect deflection that may have taken place since placing of the tile floor surface. However, no deflection was apparent to the eye nor could any be detected in walking over the floor. On the whole, this floor was in first class condition after approximately four years of service of the class for which it was designed.

SUMMARY OF RESULTS

1. Under a load equal to the design load, cracks were found in the upper surface of the slab along the edges of beams which lay between loaded panels. These appeared to be old cracks and it cannot be stated positively that any cracks were present at that time which were produced by the test.

2. Under a load equal to twice the design load there were cracks in the upper surface of the slab along all of the walls and beams adjacent to the loaded panels. At this time there were numerous cracks in the lower surface of the slab, most of these being near the centers of the panels.

3. The maximum deflections measured at the centers of the three loaded panels were 0.58 in. and 0.53 in. for the two corner panels and 0.37 in. for the other. These amount to 1/311, 1/340, and 1/473 of the short spans of the respective slabs.

4. Twenty-four hours after the test load had been removed the three slabs had recovered 64, 59, and 71 percent of the maximum deflections.

5. The stresses in the slab reinforcement were not proportional to the loads applied, but increased more rapidly at the higher than at the lower loads. For several of the gage lines there was a

marked acceleration of the rate of increase of stress with load at a load of about 180 lb. per sq. ft.

6. The stress at three gage lines exceeded slightly the yield point of the steel. These gage lines were at sections of negative moment in the slab.

7. At most of the gage lines in the concrete the strains measured were very small and in no case were they large enough to indicate stresses greater than 50 or 60 percent of the ultimate strength of the concrete.

8. At nearly all gage lines there were residual strains in the steel and concrete after the load was removed. There seemed to be no close relation between the amount of these residual strains and the maximum strains developed under load.

9. The moment coefficients $\left(\frac{Af_s jd}{w_1}\right)$ for the slabs computed from observed tensile stresses were somewhat lower than those for the Waynesburg slab, reported in Technologic Paper No. 220 of the Bureau of Standards. On account of the fact that the stress increased more rapidly than the load it would be expected that the moment coefficient would increase as the stress increased. Carrying the test farther as was done with the Waynesburg slab probably would have resulted in larger moment coefficients.

10. Due to the difference in depth of embedment of the reinforcement at the edge and at the center of panel 7A the provision for negative moment at the first interior support in the direction of the short span was only 0.58 of the resistance to positive moment at the center of the span assuming equal stresses at the two places. However, the average stresses at the edge of the slab were enough greater than those at the center of the span to result in a negative moment 1.10 times as great as the positive moment. Similarly, the resistance provided at the wall support was 0.36 of the resistance at the center of the panel while the moment based on the average observed stress at the wall was 0.76 of that at the center of the panel. In both cases the ratio of negative to positive moment was approximately twice as great as was provided for in the construction.

11. After approximately four years of service the floor of this building is in first class condition. Though a few cracks are

present in the tile surface of the floor, the cracks are less prominent than those frequently seen in the highest class floors. Without minute examination all the cracks would escape detection. There is no visual evidence that the floor has deflected since it was put into service.

Readers are referred to the JOURNAL *for May 1930 for discussion which May develop. Such discussion should reach the Secretary by April 1, 1930.*

ABSTRACTS

of the current literature of cement and concrete

PREPARED UNDER THE DIRECTION OF THE
AMERICAN CONCRETE INSTITUTE
COMMITTEE ON ABSTRACTS

J. C. WITT, Chairman, (*Materials*)
H. F. GONNERMAN (*Properties of Concrete*)
F. E. RICHART, (*Engineering Design*)
REXFORD NEWCOMB (*Architectural Design*)
THEODORE CRANE (*Field Construction*)
W. D. M. ALLAN (*Shop Manufacture*)

EDITED BY DWIGHT F. JENNINGS

ABSTRACTORS THIS MONTH

F. O. ANDEREGG, Senior Industrial Fellow, Mellon Institute of Industrial Research, University of Pittsburgh, Pittsburgh, Pa.

C. BACHMANN, Editorial Dept., *Concrete*, Chicago, Ill.

A. E. BEITLICH, Whiting, Ind.

H. FRAUENFELDER, Algonite Stone Mfg. Co., St. Louis, Mo.

P. J. FREEMAN, Chief Engineer, Dept. of Public Works, Bureau of Tests and Specifications, Pittsburgh, Pa.

H. J. GILKEY, Associate Professor of Civil Engineering, University of Colorado, Boulder, Colo.

A. J. HOSKIN, Editor, *Pit and Quarry*, Chicago, Ill.

F. H. JACKSON, U. S. Bureau of Public Roads, Washington, D. C.

DWIGHT F. JENNINGS, American Concrete Institute, Detroit, Mich.

V. P. JENSEN, University of Illinois, Urbana, Ill.

J. J. LANDY, Associate Editor, *Rock Products*, Chicago, Ill.

D. E. LARSON, University of Illinois, Urbana, Ill.

H. WALTER LEAVITT, Testing Engineer, University of Maine, Technology Experiment Station Testing Laboratories, Orono, Maine.

REXFORD NEWCOMB, Professor of the History of Architecture, University of Illinois, Urbana, Ill.

F. E. RICHART, Research Associate Professor of Theoretical and Applied Mechanics ,University of Illinois, Urbana, Ill.

N. H. ROY, University of Illinois, Urbana, Ill.

C. R. YOUNG, Professor of Civil Engineering, University of Toronto, Toronto, Ont., Canada.

ROY N. YOUNG, Chemical Engineer, Lehigh Portland Cement Co., Allentown, Pa.

MATERIALS

Properties of cements at different consistencies of mortar and concrete. H. RICHARZ. *Tonind. Ztg.* (Germany) 1929, V. 63, 1619-21, 1636-8, 1658-60.—The several cements tested (except alumina cement) behaved much the same in wet consistencies as in earth-dry mixes, but the decrease in strength was considerable even up to ages of one year. The further addition of finer aggregate beyond a certain point, increased greatly the water requirement for any given consistency. The early strength of earth-dry mortar and concrete is not reached in 6 months by wet mixes. A tamped 1:7 concrete is always stronger than any 1:3, 1:4 or 1:5 concrete wet enough to pour. Increasing the cement content helped appreciably, however, for the wetter consistencies. With suitable care, pouring of concrete is feasible for many purposes.—F. O. ANDEREGG

The bonding of Rhenish pumice stones with lime and cement. WILLI SERKIN. *Tonind. Ztg.* (Germany) 1929, V. 33, 1511-2, 1542-3.—A pumice stone deposit along the right bank of the Rhine is described as well as experiments which showed that lime has nearly as good cementing properties as cement; best results were with a mixture of lime and cement. In this way a large line of cement products is manufactured.—F. O. ANDEREGG

Powdered admixtures in concrete. *Eng. News-Record*, Nov. 21, 1929, V. 103 No. 21, p. 816.—A resume of a paper describing two series of tests reported by Duff A. Abrams at meeting of Am. Soc. Testing Materials on effect of powdered admixtures on strength and workability of concrete. Tests with commercial and non-commercial admixtures in percentages ranging from 2 to 15 per cent of the cement, indicated that slightly hydraulic admixtures showed only small increase in strength and non-hydraulic admixtures showed reduction in strength up to about 5 per cent for each 1 per cent of admixture in terms of weight of cement. Colloidal clays and diatomaceous earth gave greatest reduction in strength and in general reduction was greater at one to three days than at greater ages. Workability was not improved by admixtures. The following conclusions by the author are given: (1) The water factor (defined as a quantity of mixing water required by the addition of a unit weight of any constituent in order to maintain a given workability) required to produce the same workability as similar concrete without admixture was approximately proportional to the normal consistency of the admixture as measured by the Vicat needle; (2) all of the commercial admixtures tested must be classed as injurious adulterants, because of their high water factors; (3) the effect of admixtures was independent of the quantity of admixtures used, although quantities higher than 15 per cent were not considered. (cf. *Proc. Am. Soc. Testing Materials*, 1929, V. 29, Part 2, p.618 653.) —D. F. JENNINGS

Diatomaceous earth as admixture for concrete. WERNEKKE. *Tonind. Ztg.* (Germany) 1929, V. 53, 1536-7.—Description of the properties of Celite, which is now available in Germany.—F. O. ANDEREGG

The influence of magnesium fluosilicate on portland cement. *Zement* (Germany) 1929, V. 18, p. 1302-4.—Concrete test pieces were treated with $MgSiF_6$ and the following changes in the properties found: increase in weight up to 7 per cent, increase of the hardness and the capability of resistance against organic acids and decrease of the permeability for water. Petrographic examinations showed a denser structure of the concrete.—A. E. BEITLICH

The importance of the fineness and the chemical composition for the need of water of cements. HANS KUHL. *Zement* (Germany) 1929, V. 18, p. 1322-8.—Cements of different chemical composition were ground to different fineness. Test pieces were made (1) of normal consistency, (2) with 40 per cent and (3) with 70 per cent excess water. The strength properties were studied. The amount of water for normal consistency increases with the fineness. The chemical composition showed no influence on the

relation between strength and the w/c ratio. The strength decreases with the excess of water, especially in the finer portions. These did not develop the greatest strength. The amount of water and the strength properties depend on the presence of coarse and fine material, since both are necessary for the hardening process.—A. E. BEITLICH

The influence of the heat in the finishing mill on the setting time of cement. *Zement* (Germany) 1929, V. 18, p. 1348-50.—A high temperature in the finishing mill does not affect the clinker but partly dehydrates the gypsum. The cement has quick setting properties when mixed with water in the usual way, but it sets normal, when mixed for about 15 minutes.—A. E. BEITLICH

Tricalciumsilicate. ERNST JANICKE. *Zement* (Germany) 1929, V. 18, p. 1345-7.—Author claims the X-ray photograph of the tricalciumsilicate, made by Hansen and Brownmiller shows a mixture of dicalciumsilicate and free CaO. A. Guttmann and F. Gille dispute this conception. X-ray diagrams are given. —A. E. BEITLICH

Storage bins in the cement industry. OTTO SCHOTT. *Zement* (Germany) 1929, V. 18, p. 1371-5.—A number of modern storage bins for raw and finished materials are described.—A. E. BEITLICH

Experiments to increase the efficiency of compeb mills. TH. TRAMPE. *Zement* (Germany) 1929, V. 18, p. 1299.—Effiicencies of raw mill, a coal mill and a cement mill were increased by changing the charge of balls in the different compartments. Balls of different diameter, cubes and cylinders (Cylpeps) were used. Tables showing the different combinations of the charge, the power consumption and the fineness are given.—A. E. BEITLICH

Influence of aluminium on mortar strength. H. W. LEAVITT, J. W. GOWEN AND L. C. JENNESS. Maine Technology Exp. Sta., Paper No. 8, *Proc. Nat. Acad. Sci*, Sept. 1929, V. 15, No. 9, p. 740-42.—A statistical analysis of the 7-day and 28-day tension data of standard briquets made from 117 native glacial sands of Maine, proves that the per cent of aluminium present in the sand affects the resulting strength. This influence of aluminium content is positive—the greater the per cent of aluminium present the greater the resulting strength. The effect of this element on the 7-day strength is not as great as for the 28-day period. This would indicate that aluminium is a desirable constituent of sands. Aluminium was found to have very little effect upon the compressive strength of these sands when tested in a 1:2 mix and made into 2-in. by 4-in. cylinders. (cf. Influence of Iron Content on Mortar Strength, Leavitt and Gowen, *Proc. Nat. Acad. Sci.*, April 1927, V. 13, No. 4, p. 263-65.)—H. W. LEAVITT

On the joint influence of iron and aluminium in native sands on mortar strength. H. W. LEAVITT, J. W. GOWEN AND L. C. JENNESS, Maine Technology Exp. Sta., Paper No. 9, *Proc. Nat. Acad. Sci.*, Sept. 1929, V. 15, No. 9, p. 742-43.—Previous papers have shown that both iron and aluminium influence mortar tensile strength. This paper deals with the joint influence of these two elements, common to Maine glacial sands. A multiple correlation study is made of the data on 117 sands and a prediction equation is derived—Mortar Strength = 20 Aluminium % + 43 Iron % + 272.

The results obtained furnish proof of the fact that iron and aluminium jointly and separately materially influence the tensile strength of Maine sands. This conclusion may be extended to the chemical condition of the sand used in cement mortars to the extent of influencing the tensile strength of these mortars when hardened. Proof is thus furnished of the chemical interaction of the sand and cement. Reference is made to 2 previous papers. (cf. Gowen and Leavitt, *Proc. Nat. Acad. Sci.*, April 1927, V. 13, No. 4, p. 263-5 and Leavitt, Gowen and Jenness, *Proc. Nat. Acad. Sci.*, September 1929, V. 15, No. 9, p. 740-2.)—H. W. LEAVITT

A method of studying the reactions in a portland cement kiln. WILLIAM N. LACEY AND HUBERT WOODS. *Ind. Eng. Chem.* 1929, V. 21, No. 11, p. 1124-1126.—The kiln charge

is sampled throughout the length of the kiln during operation. Near the discharge end samples are taken by a specially designed sampling cup. At other points samples are obtained through tubes inserted in the kiln wall. Loss on ignition and free lime content determinations furnish data concerning the progress of reactions. Results of analysis of two sets of samples and sampling locations are shown in graphs. —ROY N. YOUNG

Three Forks Portland Cement Co.'s Trident plant was first in Montana. F. C. LINCOLN. *Pit and Quarry*, Nov. 6, 1929, V. 19, No. 3, p. 39.—This plant was erected in 1909 as a subsidiary of Ideal Cement Co. Its original daily capacity was 1,500 bbl. with two kilns but another kiln was added in 1913 increasing its capacity to 2,250 bbl. Is 6 mi. northwest of Logan, Mont., on main line of Northern Pacific Ry. Raw materials are Three Forks shale, an argillaceous limestone running about 70 per cent $CaCO_3$, and the Madison limestone which runs about 85.5 per cent $CaCO_3$. Quarry face is about one-half mile long with maximum bank of 140 ft. Combination of well-drill holes and coyote holes for blasting. Ingenious "coyote-car" enables single workman to excavate, load muck, tram and dump, without leaving the hole. Steam shovels and locomotive handle material to primary gyratory crusher. Material then passes rotary screen, another gyratory crusher, and swing-hammer pulverizer. Ground material passes by pan conveyor to the blending bins in dry-process mill, and is automatically sampled to 20 lb. per 160-ton bin. Belt conveyors beneath the bins carry the blend to dryer bins. Two dryers reduce moisture in coal used for fuel to 0.5 per cent. Dried rock elevated by pan conveyors to grinding room. First-stage grinding is by Kominutors, second by tube-mills. Screw conveyors deliver ground material to the three kilns, each 140 ft. long. Clinker goes by conveyor to two 60-ft. coolers and then to same grinders that prepare the feed. Five concrete bins, capacity 40,000 bbl., store finished cement. Two 3-tube packers sack it.—A. J. HOSKIN

Slurry having both abrasive and corrosive properties can be pumped efficiently. *Pit and Quarry*, Nov. 6, 1929, V. 19, No. 3, p. 71.—Slurry handled by Structural Gypsum Co., Linden, N. J., possesses peculiar chemical and physical characteristics which have been met by special centrifugal pumps. Some slurry carries 60 per cent solids and has temperature ranging from 150°F. to 180°F.—A. J. HOSKIN

Cement and gypsum plants at Hanover, Mont., operated by Three Forks Portland Cement Co. F. C. LINCOLN. *Pit and Quarry*, Nov. 20, 1929, V. 19, No. 4, p. 27.—Two unlike plants secure raw materials from adjacent formations. Cement-shale quarry has 50-ft. face. Rock so soft that no blasting is required. Steam shovels load to side-dump cars which are taken by a gasoline locomotive to primary gyratory crusher. Limestone quarry is 3 mi. from plant and has a bank varying from 40 ft. to 100 ft. Gasoline and electric motors drive well drills. About 40,000 tons of rock blasted at one time. Jaw crusher in quarry reduces limestone to minus 6-in. From bin this rock is conveyed by aerial tramway to cement plant. It is again reduced by the same primary gyratory that handles the raw shale. Raw materials are then weighed, mixed, and delivered into cement-mill bin. Water is added to this mixture as it enters the Kominutor, from which the material goes to and through tube mills, emerging at a fineness such that 88 per cent will pass 200-mesh. Slurries are sampled and suitably blended, using three correction tanks and three intermediate tanks, before passing to a 9 by 200-ft. kiln in which coal is the fuel. Clinker passes through 80-ft. rotary cooler, is admixed with 2.25 per cent raw gypsum, and finally pulverized in a 7 by 22-ft. ball mill. Finished product goes to storage of 80,000-bbl. capacity. Two 4-tube packers complete the process.—A. J. HOSKIN

Compressive strength of slag concrete. (See PROPERTIES OF CONCRETE.)

Calculation of the compounds in portland cement. R. H. BOGUE. *Rock Products*, Nov. 9, 1929, V. 32, No. 23, p. 47.—Arithmetical and diagrammatic methods are designed to give relative amounts of portland cement compounds from clinker analysis. Applications and limitations discussed. (cf. *Ind. Eng. Chem.*, Anal. Edition, 1, 4 (1929).—J. J. LANDY

Admixtures in concrete. JOHN G. AHLERS. *Concrete*, Dec. 1929, V. 35 No. 6, p. 21.—An analysis has been made of financial advantages in the use of sufficient admixtures, by computing the increased cost of the concrete with various admixtures and considering what might be purchased in the form of portland cement at the same cost. Comparisons were made of the concrete by the use of this additional cement with concrete using admixtures. Admixture *A* showed an average saving of about 12¢ a cu. yd; *B* 11¢ a cu. yd; due to increased yield. A loss of about 7¢ a cu. yd. is sustained when admixture *C* is used. Admixture *D* which does not swell like the three previous materials, but like cement, occupies a space equal to its own volume, showed loss of 3¢ a cu. yd. Hydrated lime causes a slight reduction in the strength as well as a loss of 11¢ a cu. yd. The use of calcium chloride involved a loss of about 23¢ a cu. yd. Admixtures in general do not compare with an equal money's worth of cement.—C. BACHMANN

In Defense of Integral Waterproofing. R. E. WITHROW. *Concrete*, Dec. 1929, V. 35, No. 6, p. 27.—It is recommended that all admixtures for concrete be classified under their general types, namely: 1st, integral waterproofings, which shall include only such materials as are water-repellents; 2nd; plasticising or lubricating materials, or those used to prevent segregation, such as lime, diatomaceous earth, or silica, etc.; 3rd, the calcium chloride combinations, which should be used as accelerators. The heading of "fillers" means nothing, but might include materials which would be actually detrimental to concrete. Any finely divided inert material used as an admixture for concrete is a filler. Densifiers may be regarded the same as fillers. In tests made by a private laboratory, of integral waterproofing, the cubes after being made were kept in the forms for two days and cured in air for five days. When seven days old the core was filled to within ½ in. of the top with a potassium permanganate solution, which was kept at a constant level for seven days, by daily adding enough solution to replace that absorbed by the concrete. Glass plates prevented evaporation. In the plain concrete core, in which no admixture was used, the solution penetrated about one-third of the thickness; in the core employing a lubricating material, fully one-half of the thickness was affected; another plain concrete core whose surface was treated with chemical floor hardener showed a penetration of about one-sixth, and the concrete in which a water-repellent type of integral waterproofing was used showed only a slightly stained surface after seven days.—C. BACHMANN

Effect of type of coarse aggregate on the strength of concrete Discussion by STANTON WALKER. *National Sand and Gravel Bulletin*, Dec. 1929, V. 10, No. 12.—An analysis of data contained in the report of W. F. Kellermann, in the June issue of *Public Roads*, presented in such a manner that comparisons among the different aggregates are made using concrete in which the quality of concrete per unit volume is constant and the volume of mortar is in proportion to the voids in the coarse aggregate. The test data which were analyzed included results of concrete tests for 4 mixtures, 4 gradings of coarse aggregate, and 14 different coarse aggregates of which 7 were designated as gravels and 7 as crushed stones. The following conclusions were drawn by the author of the discussion based on these data:

(1) Comparisons of the concrete-making properties of different aggregates should be made only in the light of careful consideration of the quantity of cement per unit of volume of concrete and the amount of mortar required to produce workability. Comparisons based on arbitrary mixtures by volume lead to erroneous conclusions.

(2) If the same quantity of cement per unit of volume of concrete and a volume of mortar in proportion to the voids in the coarse aggregate are used in the concrete, flexural strengths obtained using aggregates with rounded particles are at least as high as those obtained with angular particles.

(3) If the cement content and volume of mortar are taken into consideration as suggested in No. 2 above, the compressive strength of concrete made with coarse aggregate consisting of rounded particles generally will be higher than that of concrete made with angular particles.

(4) While the preponderance of evidence indicates rather wide variations in the flexural strength of concrete made from different aggregates it is not believed that available information definitely identifies the factors causing these variations.

(5) When the cement and mortar content are taken into account the compressive strength is found to be affected by the type of aggregate at least as greatly as the flexural strength.—P. McKIM

Fuel in the manufacture of portland cement. H. P. REID. *Rock Products*, Nov. 9, 1929, V. 32, No. 23, p. 69-72.—A critical discussion of various types of fuels, coal mill problems and practice, and fuel efficiencies in portland cement manufacturing. Temperature control in cement manufacture, care of refractory lining, fuel ratios and waste heat power generation are discussed. Recoverable heat in exit gases of average dry process kiln greatly exceeds that in modern wet process kiln. Waste heat boilers in wet process plants may not give satisfactory return on investment.—J. J. LANDY

Crushing plant of Olympic Portland Cement Co. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 49-53.—Detailed description of operation, plant design, featuring two-stage aerial tramway from quarry to plant.—J. J. LANDY

Sand and gravel in 1928. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 53.—Production and value figures by states as compiled by Bureau of Mines. About 77 per cent of total production reported as washed and screened.—J. J. LANDY

Fuller's earth in 1928. *Rock Products*, Nov. 23, 1929, V. 23, No. 24, p. 71.—Production reported by 17 operators in eight states increased 9 per cent in quantity and 3 per cent in value, compared with 1927. Exports increased 34 per cent.—J. J. LANDY

Efficient gravel washing with jigs. EDMUND SHAW. *Rock Products*, Nov. 23, 1929, V. 23, No. 24, p. 72-75.—Four-cell machine at Edward Sidebotham plant, Lomita, Calif., treats 100 tons per hour, reducing shale content to less than ½ of 1 per cent.—J. J. LANDY

Physical characteristics of slag aggregate. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 75.—Physical properties discussed. Specific gravity range determined to be from 2.11 to 3.16; weight per cubic foot, crushed and graded, 74.0 to 88.0 lb. (rodded) weight of slag concrete compared with limestone and gravel concrete. (cf. *Symposium No. 14, National Slag Association*, Cleveland, O.)—J. J LANDY

Free lime in portland cement. S. L. MEYERS. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 76-77.—Apparatus and method described to expedite and smplify usual ammonium acetate procedure.

Tricalcium Aluminate. *Rock Products*, Nov. 23.—Steel and Davey hold tricalcium-aluminate to be mixed oxide and not a salt, the original structure of neither oxide being retained. (cf. *Jour. Amer. Chem. Soc.*, 1929, V. 51.)—J. J. LANDY

Manufacture of lime and cement from gypsum. O. F. HONUS. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 78-81.—European process patents reviewed. (cf. *Zement* (Germany) 1929, V. 18, No. 37.—J. J. LANDY

Increasing the output of tube and compound mills. T. TRAMPE.

Rock Products, Nov. 23,1929, V. 32. No. 24, p. 82-83.—Changes in filling and balls in mills for grinding raw materials, coal and finished cement are given whereby efficiencies were raised.—J. J. LANDY

Developing new sand and gravel deposits. EDMUND SHAW. *Rock Products*, Nov. 23, 1929, V. 32, No. 24. —A review of economics of new sand and gravel developments. (cf. J. R. Thoenen, *Economic Paper No. 7* of the United States Bureau of Mines.) Report is for ordinary scale producer and discusses prices, prospecting, development, overburden, capital requirements and operating costs.—J. J. LANDY

Skin irritation among concrete workers. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 99.—Causes include exposure of skin to abrasive action of fine hard particles of cement and burning delicate skin by caustic solution formed by moisture and soluble lime in cement. Recommended protective salve comprises 3 parts vaseline and 1 part lanoline to prevent skin from cracking.—J. J. LANDY

Portland cement output in October 1929. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 106.—Detailed statistics by U. S. Bureau of Mines covering stocks of finished cement, production and shipments from various mills in the United States. Production showed decrease of 4.6 per cent and shipments a decrease of 5.8 per cent as compared with Oct. 1928.—J. J. LANDY

PROPERTIES OF CONCRETE

Compressive strength of slag concrete. *Pit and Quarry*, Nov. 20, 1929, V. 19, No. 4, p. 95.—(cf. *Symposium No. 18, Nat. Slag Association*, Cleveland, O.)—Some tests prove that slag concrete has strength equal to gravel or limestone concrete. One authority states that it is superior due to angular shape, rough surfaces, cellular structure of slag. Researches of numerous institutions are described. Slag-lime bricks are discussed, as also are slag mortar and lumnite cement concrete. One fact developed is that slag's porosity removes water during the mix but yields this water subsequently during the curing. A chemical similarity between slag and cement is believed favorable in the hardening. Density of slag aggregate has no effect on compressive strength of concrete.—A. J. HOSKIN

Compressive strength, density and the water-cement ratio of mortar and concrete. E. SUENSON. *Beton Eisen* (Germany) 1929, V. 28, 397-402.—If the density of the mortar or concrete is the ratio of the volume occupied by the solids to the total volume, the compressive strength is proportional to some power of this density. This gives a more general relationship than referring to the water-cement ratio, which can be applied only to plastic concrete. The method used differs from previous work (Feret or Talbot and Richart) in that the specimens were allowed to settle before observing the volume. The exponent does not seem to depend upon the method of storage of the specimens but does vary with different kinds of cement. Part of the reason for the increase in water requirement on adding coarse aggregate, lies in the boundary correction; in contact with the large surface the cement and fine aggregate do not pack as well, requiring water to fill the extra voids so produced. A method of determining absorption of sand, consists in placing 100 g. in a 500 ml. flask and adding 20 ml. gasoline and shaking thoroughly. Then NaCl soln. is added from a buret until the flask is filled to the mark, the soln., forcing any gasoline not absorbed by the sand up to the top. If the water-cement ratio varies from 0.3 to 0.7 the water in combination with the cement varies after 28 days from 18 to 25 per cent of the weight of the cement.—F. O. ANDEREGG

Experience in the physical and chemical examination of concrete. Discussion of Mr. Worsdale's paper. H. G. LLOYD AND OTHERS. *Structural Engineer*, (England) Nov. 1929, V. 7, No. 11, 419-423.—H. G. Lloyd pointed out advisability of using a cap of very thin neat plaster on compression cubes and questioned uniformity of method of filling molds and uniformity of testing speeds used in different laboratories. He stated that stone dust when mixed with cement and tested gave

different results than when it remained to be mixed with the aggregates. Harry Jackson was interested in the finding that the amount of water required permanently in concrete, in order to get the full set of cement, was in the neighborhood of 14 per cent. H. A. Holt pointed out that different stone dusts affected concrete differently, and also that he had been able to increase the strength of mortar 11 per cent by eliminating all material between the $\frac{1}{16}$ in. and the 50 mesh screens E. S. Andrews stated that a previous report had found 100 per cent variation in strength reported on identical cubes tested by various laboratories. S. Bylander pointed out the effect of dust in causing horizontal separation planes. Mr. Worsdale in his reply touched upon all the points discussed by the contributors.—V. P. JENSEN

A mathematical method of proportioning. JOSEPH A. KITTS. *Concrete*, Nov. 1929, V. 35, No. 5, p. 33.—Tables have been worked out covering constants for determining cement content for any compressive strength for a given maximum size of aggregate and slump of concrete, characteristics of job aggregates, and theoretical gradings of aggregates, as a part of a mathematical basis of proportioning. —C. BACHMANN

A substitute for the compression test of concrete. GEORGE J. GRIESENAUER. *Eng. News-Record*, Nov. 28, 1929, V. 103, No. 22, p. 846-847.—In connection with railway structures a method was used for determining the composition and quality of concrete while in its plastic state by "unscrambling" to check actual composition against contract specifications and calculated strengths against results obtained with test cylinders made at time concrete was placed. The analysis is made before concrete hardens, by washing cement from a weighed sample of the mix, drying, pulverizing and sifting. Dried sand and gravel are screened and weights of the three materials accurately determined. Weight of original water content of sample is found by subtraction. Analysis values are reduced to basis of one sack of cement and strength as determined from Abrams' curve for W/C ratio found by the analysis is checked against the actual test values. An illustrative case is worked out and followed by table comparing strengths by test with values determined by analysis.—D. F. JENNINGS

Properties of cements at different consistencies of mortar and concrete. (See MATERIALS.)

Powdered admixtures in concrete. (See MATERIALS.)

The influence of magnesium fluosilicate on portland cement. (See MATERIALS.)

ENGINEERING DESIGN

Classification, selection and adaptation of high dams. D. C. HENNY. *Proc. Am. Soc. Civil Eng.*, Nov. 1929, V. 55, No. 9, p. 2327-36.—Because of tendency to build dams of ever-increasing height, foundation requirements, conditions limiting height, and sources of danger become increasingly important. Leading dams of various types and their height are mentioned in the paper. The complete rejection of a dam site is usually based upon economical considerations since there are few sites upon which some form of safe dam could not be built. Masonry dams because of their rigidity make greater demands upon the foundation than do the relatively flexible earth and rock-fill dams. Increased height will necessitate added care in the selection of sites and the use of stronger mixtures and higher design stresses. Possibilities are not exhausted in either field. Other requirements are water tightness of foundation and structure. The latter can be controlled by the richer mixtures and care in bonding but the foundation is least under control and is likely to be the element that will fix the safe limit of height. Compared with a gravity section, the buttress type (multiple arch, multiple dome, or Ambursen type) taxes the foundation much less. Foundation pressures and uplift can be more easily controlled. Foundation tightness requirements are about the same as for the gravity dam. The single arch dam is limited to narrow rock walled

sites. Secondary tensile and compressive stresses at the base are sometimes difficult to control. The introduction of horizontal expansion joints, reinforcement where needed, or the dome principle have been used or suggested. Because of the stress effect that results from a slight yielding of the abutments the arch dam makes greater demands upon them. The results from the Stevenson Creek Test Dam and from model tests (cf. *Proc. Am. Soc. Civil Eng.*, May 1928, Part 3) have contributed to a better design understanding of the arch dam. The danger of damage from flood overflow is not usually great for a masonry dam and is less for the arch than for other types because of its decreased dependence upon toe stability. Very severe earthquake shocks have been borne by nearly all types of dams but the arch gravity masonry dam is deemed safest. Dams across faults require special consideration. Shrinkage cracks from cooling are common in concrete dams. The temperature range from setting heat is 30°F. or more while 5°F. will stress concrete to over 150 lb. per sq. in. in tension. Drying out also causes shrinkage. In gravity dams transverse cracks are usually offset in part by introducing contraction joints. Cracks parallel to the axis whether in the mass of a gravity dam or in the buttresses of other types are likely to be close to the direction of maximum shear and may also become equivalent to a dangerous narrowing of base or section width. In buttresses it is often possible to introduce sufficient reinforcement to prevent such cracks, but this would be very costly in gravity dams. Shrinkage cracking can be lessened by giving the concrete more opportunity to dissipate its chemical heat by slowing down the rate of concreting, pouring in thinner lifts (4 ft. or less), keeping the concrete wet between pours, and sometimes by precooling the mixing water and aggregate. If the dam is kept at about the same elevation over its length, expansion and therefore the ultimate shrinkage will be held down by longitudinal thrust. While not a great deal is known about the extent or distribution of uplift, in masonry dams, the usual precautionary measures should be taken. Tension in the foundation at the heel has had little attention. It increases with the height of dam, however, and will probably require precautionary measures such as the placing of a triangular body of plastic clay upstream along the heel.—H. J. GILKEY

The new waterworks of the city of Prague. F. KLOKNER, EM. SNIZEK AND B. HACAR. *Beton Eisen* (Germany) 1929, V. 28, 389-96.—General description.—F. O. ANDEREGG

Handbuch fur Eisenbetonbau. (Germany): Wilhelm Ernst & Son. R. M. 6.80. Reviewed in *Zement* (Germany) 1929, V. 18, p. 1314.—A. E. BEITLICH

Concrete reservoirs and tanks. *Concrete Constr. Eng.* (England), Nov. 1929, V. 24, No. 11, p. 653-658.—Principles of design of open and covered reservoirs and tanks are given. The design of side walls is treated by the usual methods used in retaining wall design. Special conditions are stated for tanks of different shapes.—F. E. RICHART

Reinforced concrete warehouse at Bethnal Green. *Concrete Constr. Eng.* (England), Nov. 1929, V. 24, No. 11, p. 653-658.—The new 7-story dairy warehouse building at Bethnal Green is built of reinforced concrete throughout. The floors were designed for loads of 2, 3, and 4 cwt. per sq. ft. The column spacing is 26 ft. in both directions. Beams and girders serve to cut the floor slabs into panels 13 ft. square. The design includes cantilever footings at junction with adjacent building and heavy shear reinforcement in beams carrying heavy floor loads. Special truck loading platforms or docks are provided at the ends of the building. The exterior of the building which is entirely of concrete cast in place, presents a striking and pleasing appearance.—F. E. RICHART

Reinforced concrete design simplified. JAS. R. GRIFFITH. *Concrete*, Dec. 1929, V. 35, No. 6, p. 37.—Chart E_1 covers the design of web reinforcing. It is another one of the series which are

now based on the A. C. I. building code. (cf. Tentative building regulations for reinforced concrete, *Proc. Am. Concrete Inst.*, 1928, V. 24, p. 791.—C. BACHMANN

Notes on the foundations of a new Thames-side warehouse. T. F. BURNS. *Structural Engineer*, (England) Nov. 1929, V. 99, No. 2, p. 431-440.—Warehouse for Messrs. Oxo, Ltd. at Blackfriars utilized a portion of the old G. P. O. Power Station which was demolished. Footings embodied following features: (1) combined trapezoidal footing consisting of an inverted reinforced T-beam to carry interior and exterior columns, (2) combined footing at corner of site consisting of an inverted T-beam and an L-beam connected by two stablising beams (part of computations given for this case), (3) a series of cantilever beams 4 ft. 3 in. wide by 5 ft. deep carrying columns at the ends and having an interior fulcrum support of an inverted T-beam running beneath the existing concrete floor, (4) a continuous beam over seven old bases with the ends cantilevered and with the beam carried down to firm bearing between the old bases by means of an inverted T-section, (5) addition to old mass footings by adding a block alongside them, (6) new footings adjacent to old ones in which bonding was provided for by hacking, wetting, and grouting, and in which freedom of movement was provided for by the use of clay joints. A resume is given of the problem of proper consistency.—V. P. JENSEN

Notes on concrete arch of unusual design; aids highway relocation. D. M. MCPHETRES. *Eng. News-Record*, Nov. 21, 1929, V. 103, No. 21, p. 815-816.—Highway bridge recently completed by Santa Cruz County, California, employs wide arch ribs eccentrically loaded to support spandrel columns on curve. New bridge is 326 ft. long, curved to a center line radius of 400 ft. Main span consists of a 123 ft. two-ribbed spandrel arch. Two ribs, 23 ft. 9 in. center to center, 2½ ft. deep at crown, are 7 ft. wide in order to provide for the curved alignment of super structure columns. The two-column bents are spaced at 15 ft. centers, are all parallel, and are at right angles to the center line between the arch ribs. The outside faces of all columns, are parallel to and 13 ft. from the roadway center line curve. This design feature was adopted instead of a structure with columns in a straight line and with sidewalk brackets of varying length, because of the better appearance of uniform overhang. The deck has a 24 ft. roadway with a superelevation of 2 ft. and two 4½ ft. sidewalks. The height of roadway above stream bed is 72 ft. The arch has a clear rise of 38 ft. 6 in.—N. H. ROY

Notes on precast concrete bridge railing; design allows future shift. *Eng. News Record*, Nov. 28, 1929, V. 103, No. 22, p. 847.—Design by the California Division of Highways allows for future widening of roadway. The present highway bridge over the Salinas River consists of ten 100-ft. steel deck spans and 17 reinforced concrete spans 37 ft. long, all 24 ft. wide between curbs. Design provides for future 5 ft. cantilever section to be added on each side. This necessitated a movable railing. A railing was designed for 1000 lb. per lineal foot impact, to be used on steel spans as well as on concrete spans. It has an L-shaped base that provides the curb, and place for anchor bolts. Alignment of railing at edge of slab will be made with cast iron wedges driven under the base in wedge pockets and grouted in. Grout holes in base permit grouting between base and slab. Four bolt holes in the sides of the base of each 10-ft. section of railing permit anchorage to slab. One drainage hole under the edge or curb portion of rail and through the slab is provided every 20 ft. To widen the bridge, 5 ft. cantilever splices will be added to floor beams on each side, new slab will be poured, and railings moved over 5 ft. and fastened as before, all without interruption to traffic. The design was prepared in the office of C. E. Andrew, Bridge Engineer, Division of Highways, California Department of Public Works.—N. H. ROY

Building high-head siphons on the Yakima project. *Eng. News Record*, Nov. 14, 1929, V. 103, No. 20,

p. 763-765.—A part of the construction program being carried out by the U. S. Bureau of Reclamation on the Yakima Project consists of the building of 25 inverted siphons 11 of which are required in the 26-mile main canal of the Kittitas division. The large size of the canal and the irregular topography made it desirable to cross the waterways with inverted siphons instead of continuing on grade around them. The siphons are about 12 ft. in diameter and are cast as monolithic reinforced concrete. One of these is subject to a head of 153 ft. For that portion of the barrel subject to less than 100 ft. of head a 1:2:3.25 mix with 1½ per cent admixture of diatomaceous silica was used. For barrels subject to heads in excess of 100 ft. a 1:1.7:2.8 mix without any admixture was used. It was found by experiment that the addition of diatomaceous silica to the leaner concretes resulted in a marked improvement in workability and permeability but did not improve the richer mixes. The inner form used in the construction of the barrel resembled a wood stave pipe and was supported by steel bars projecting from concrete piers spaced on 8 ft. centers. This was necessary because ties were not permitted to pass through the shell. Great care was given to the construction of joints between successive pours. All joints were normal to the axis of the barrel thus requiring the use of bulkhead forms between the inner and outer barrel. Each joint was provided with a copper water stop having a ¼ in. crimp to allow for expansion and contraction of the joint. The copper strips were coated with an emulsion of asphalt in water to prevent leakage along the line of contact between concrete and copper. The joints occur at intervals of about 30 ft., an average day's pour.—D. E. LARSON

Lining canals with concrete on the Yakima project. *Eng. News Record*, Nov. 7, 1929, V. 103, No. 19, p. 722-725.—One of the major problems encountered in the construction of the Kittitas Canal of the Yakima Project of the U. S. Bureau of reclamation was that of providing drainage for the 13 miles of concrete lined section. Longitudinal drains of bell-and-spigot clay or concrete pipe covered with gravel were laid with open joints and vented through outlets at intervals of 300 to 800 ft. The joints of the outlets passing through the lower canal bank were cemented. Provision for diverting surface water was made by sinking cut-off walls in the swales above the canal and constructing concrete troughs to discharge the water over the lining into the canal. In spite of these precautions, groundwater pressure sufficient to crack the concrete developed in some places and had to be relieved by weep holes. These consisted of pipes grouted into the lining and threaded for capping when the canal is filled. The form used for placing concrete on the side slopes of the canal was a steel panel 3½ ft. by 14 ft. supported at each end by light steel trusses in such a manner that it could be moved up or down the slope as the work progressed. A crew of three men working on this panel tamped the concrete into place beneath it and moved it when necessary. The proper thickness of concrete was secured by adjusting the jacks at the ends of the trusses. Method of tamping was found to be highly satisfactory, particularly because of greater degree of impermeability thus secured.—D. E. LARSON

Notes on rational design of concrete. ARY F. TORRES. Polytech. School of Sao Paulo, Brazil. *Bull. No. 3 Materials Testing Laboratory*, (Brazil) June 1929, 32 pages.—Investigations on plain concrete were made by this laboratory from 1926 to 1929 and information presented in a paper before the Second Pan-American Highway Congress. Many of the factors affecting concrete and its design are discussed under the following headings: (1) empirical methods, (2) rational design, (3) strength of paste, or neat cement, and method of obtaining any desired strength, (4) strength of concrete, (5) plasticity of concrete and how to control it, (6) a rational method of design of concrete mixes evolved from laboratory experiments and from practical applications

by cooperating constructors of Brazil, (7) numerical examples illustrating this proposed design of mixes. Reference is made to the work of Abrams, Graf, McMillan and others. The "Rational Design of Concrete" is executed in the following manner: The desired strength is chosen and increased 10 per cent to cover variations in the strength of cement, and the water-cement ratio is read from an experimental curve. A tentative choice of m (weight of mortar per kilo of cement) is made. A value of n is chosen, where n is the ratio of weight of coarse aggregate mortar (apparently meaning a mixture of coarse aggregate, cement and water) to weight of fine aggregate mortar. With these values of m and n, the weight of sand is found from another diagram, thus determining tentative proportions. Moisture contents of the sand and gravel are determined by Chapman's method, and used to correct the weights of materials in the tentative design. If this tentative design is not workable, the values of m and n only are varied until a workable mix is obtained. The new idea in this method of design is the employment of a chart showing the relation between weight of sand and weight of mortar, for given values of the ratio of "coarse aggregate mortar" to fine aggregate mortar. Ten figures are given in the bulletin, three of which are used in design. These show: (1) Relation between the moisture content of sand and its own change in volume, (2) Strength of concrete (or of neat cement paste) as a function of the volume of water per kilo of cement, (3) Strength of concrete as a function of age, (4) Ratio of weight of sand to weight of mortar for various ratios of weight of coarse aggregate mortar to weight of fine aggregate mortar, and (5) A nomographic chart showing water-cement ratio, ratio of weight of mortar to weight of cement, and weight of cement per cubic meter of concrete. Translated from Portugese.—N. H. ROY

Details of steel cores for reinforced concrete columns. FRANK A. RANDALL. *Concrete*, Nov. 1929, V. 35, No. 5, p. 13.—Column cores of H-section are invariably used. As the column load increases heavier sections are employed, but of the same nominal size, so that the outside dimensions of the concrete encasing may remain practically unchanged. In the typical column base the base plate consists of a square of rectangular steel plate set on cement grout approximately one in. thick. The area of the base plate, in sq. in., is obtained by dividing the column loads (in lb.) by 600. The end of the steel H-section is milled and is fastened to the plate by two angles. Four anchor bolts are provided, extending from 24 to 30 in. into the concrete pedestal. Aside from the anchor bolts there is no other connection between the column and the base plate. In wall columns and corner columns two straight bars in the top and in the bottom of wall girders are used. The two top bars are passed through the two upper holes in the steel column section, while the bars from the opposite direction are passed outside the steel column section. The two straight bars in the bottom of the wall girder are passed through the two lower holes, and the straight bars from the opposite girders are passed along the side of the steel section.—C. BACHMANN

Form work details for flush beam jobs. (See FIELD CONSTRUCTION.)

Why joints in concrete pavements. (SEE FIELD CONSTRUCTION)

ARCHITECTURAL DESIGN

Modernism in the Alps. *Architect and Building News* (England), Nov. 1 1929, V. 61, No. 11, p. 537-541.—Two new concrete hotels in Murren, Switzerland, the work of Arnold Jtten, Architect, are featured. These hostelries attempt to solve the hotel problem and do not attempt "to produce beauty for its own sake, but as the outcome of a comprehensive study of this particular problem. These hotels, with their clear bright interiors, their sunny rooms, and wide balconies, their furniture with smooth surfaces and simple profiles, are not forbidding or unhomelike. They have an aesthetic attraction of their own; they produce the not unpleasing impression of kinship with the yacht or the limousine. Above all, their architecture is free from triviality."—REXFORD NEWCOMB

Cooling towers. *Architects' Journal* (England), Nov. 6, 1929, V. 70, No. 1816.—Frontispiece shows the concrete cooling towers of the New Birmingham Power Station. Constructed throughout of concrete, 168 ft. in diameter and 210 ft. high, they have the form of great concave truncated cones. They are representative of the varied functional forms that are creating a new spirit in architecture.—REXFORD NEWCOMB

Oak Hill mausoleum, San Jose, California. ALBERT F. ROLLER. *Architect and Engineer*, Nov. 1929, V. 99, No. 2, p. 51-53.—The building is of heavy reinforced concrete construction. The main walls, floors, and roof were designed independent of the crypt structures within. The crypt blocs are supported on footings separate from the building foundations and every attention was given by H. J. Brunnier in the structural design to make the crypts as secure as possible from any damage by earthquakes or by the elements. No wood was employed in the permanent construction of the building and wherever exposed metals were found necessary, only copper or bronze was used. The building, of Romanesque influence, is approached by wide level roadways through attractive plantings. On entering one passes through a rotunda 30 ft. in diameter and 52 ft. high, dedicated as a chapel where committal services may be held. The exterior of the building is executed in cast stone and cement plaster. The niche rooms on the columbarium floor encircling the rotunda are clad in marble and the niches constructed of cast concrete are faced with marble and plate glass set in frames of cast bronze. A constant circulation of fresh air has been provided for each individual crypt by means of a 1¾-in. inlet at the bottom of one end of the crypt and the same size outlet at the top at the opposite end. These vent pipes were cast into the reinforced concrete crypt walls, the inlets extending 6 in. below the bottom of the floor beams and pulling air from the excavated area under the building. The outlets extend into a plenum chamber formed by the space between the top of the crypt tiers and the roof slab and from which the air is drawn by suction ventilators.—REXFORD NEWCOMB

Los Angeles power and light plants. FRED L. ROEHRIG. *Architect and Engineer*, Nov. 1929, V. 99, No. 2, p. 75-76.—The city of Los Angeles possesses the largest municipally owned electric utility in the United States; and moreover the city's electrical equipment is housed in structures befitting the pride manifested in her churches, libraries, and other civic buildings. Apparently it has been the aim of the Department to make its buildings not merely a housing for electrical equipment, but that they shall have an architectural expression of dignity and repose in keeping with their function, and at the same time be an aesthetic asset to the neighborhood in which they stand. To this end the Department has selected designs of simple lines, good proportions and employing only one constructive principle of square headed openings. Construction of these buildings is reinforced concrete walls and floor slabs, or steel frames enclosed in brick-work. The exterior of San Francisquito Power Plant No. 2, a hydro-electric generating station 43 miles north of Los Angeles, recently finished, is designed along lines showing a vertical treatment accentuated with bold ornament in keeping with the ruggedness of the canyon in which the building stands. The plan of station B, a receiving station is such as to call for an exterior of horizontal masses. The walls are dressed with a surface grinder, still leaving the form marks to give surface texture.—REXFORD NEWCOMB

Finished concrete surfaces. *Architectural Forum*, Nov. 1929, V. 51, No. 5, p. 578-80.—The finishing of concrete surfaces falls naturally into two classes: (1) for wearing surfaces and, (2) for all other exposed areas. Each may be of character, composition, texture and color to suit the usage or purpose for which it is intended, or to satisfy the idea of the designer. There appears to be an increasing use of monolithic surfaces . . . many architects even leaving the concrete entirely untouched, after the forms are re-

moved. When this course is to be taken, one must make sure that the form work is in exact shape to produce the desired results, especially that the lines of demarcation between successive pourings do not appear in undesirable places. Sometimes false joints are called for, and strips are nailed inside of the forms to produce such an effect. All construction joints should then be made to occur at these strips and are thus rendered invisible in the finished work. This is really imitation stone, not often called for. Such design is more frequently executed in pre-cast blocks. Although the use of exposed concrete surfaces is apparently increasing, there are still many designers who specify the application of cement plaster or stucco to the rough concrete, especially if some unusual color or texture is sought. The chief objection to exterior plastering on concrete is the prerequisite of extreme care and expert workmanship, lacking which the finished material may soon show fine cracks and eventually spall off. This is especially true where it is subjected to freezing. Since exterior stucco is chiefly used as a cheap covering for tile walls and their imitation done in lath and studding, it would appear to be wasted in the disguise of more permanent construction. Ornament and run-molds, frequently used in connection with exterior plastering need very close supervision, whether pre-cast or worked in place. Ornament cast in place demands the most careful manipulation of the flux in the forms, use of all coarser aggregate being entirely eliminated. The "waste molds," in which the actual ornament is formed, are first submitted for approval, and hence one is thereafter concerned only in the mechanical process of filling the forms.—REXFORD NEWCOMB

Catalog, British Portland Cement Association, Ltd. *Architectural Review* (England), Nov. 1929, V. 66, No. 396, p. 76.—During the last decade great advances have been made in concrete construction. America has led the way. The new catalog of the British Portland Cement Association, Ltd., by picturing buildings in America shows how American architects are taking advantage of the great possibilities of concrete. Illustrations show buildings in California, where the architecture is Spanish in type. Perhaps the most striking among the illustrations of the varied adaptations of concrete are those which show its use in ornamentation. These molded decorations, after the first reproduction from the molding, can be repeated at so low a cost that they no longer represent a very expensive item, a fact of which the American architects appear to have availed themselves. Illustrated by: Elk's Club, Los Angeles, Curlett and Beelman, Architects, and Beverly Professional Building, Beverly Hills, Harry E. Werner, Architect.—REXFORD NEWCOMB

FIELD CONSTRUCTION

Form work details for flush beam jobs. C. W. ELLSWORTH AND H. S. KELLER. *Concrete*, Nov. 1929, Vol. 35, No. 5, p. 21.—The bottom of the beam is at the same elevation as the bottom of the floor joists. The ribbed floor was 12 in. plus 2½ in., making beams 14½ in. deep. Because all the beams were 14½ in. deep, all the shores were of uniform length. In erecting forms, procedure was as follows: First, the shores were erected and 2 by 10 in. stringers spiked to them. With the exception of a few bays the joists ran at right angles to the longest side of the building and these joists were carried continuously through all the bays where the joists were parallel. There were no beam sides as the ends or sides of the pans formed the beam, as the case might be.—C. BACHMANN

Repairing floor finish. *Concrete*, Nov. 1929, V. 35, No. 5, p. 27.—Cracks in concrete floor finishes may be classified under two headings: (1) Cracks originating in the base and extending through the finish, commonly called structural cracks. (2) Cracks confined to the finish layer, further divided into (a) cracks extending through the full thickness of the topping, and (b) those of a superficial nature, ordinarily called hair cracks, or crazing. Chipping off defective topping should be carefully done with pneumatic tools or by hand, and the entire area should be removed until clean, sound concrete is exposed.

Depth should never be less than one inch. Feathered edges should be avoided and the full depth of the chipped area maintained up to the point where it joins with the sound topping. Patches should be protected until they have been thoroughly hardened and cured.—C. BACHMANN

Concrete mixing plant erected for use during erection of power house. F. A. WESTBROOK. *Pit and Quarry*, Nov. 20, 1929, V. 19, No. 4, p. 58.—Central concrete-mixing plant (described) was erected for temporary use by New York Edison Co. in New York, N. Y. Equipment operated night and day for more than one year with very little wear.—A. J. HOSKIN

Albany Trucking Corp. central mixing plant at capacity in first year of operation. *Pit and Quarry*, Nov. 20, 1929, V. 19, No. 4, p. 79.—Sand and gravel, purchased from a neighboring producer, received by trucks which dump materials into a hopper feeding an inclined belt conveyor to top of the plant and are diverted, as desired, into four compartments of a steel bin. Crushed stone is received by rail, dumped through track and three gates to another inclined belt conveyor which discharges to a bucket elevator to the steel bin. Compartments of the bin contain $\frac{5}{16}$-in. to 1-in. gravel, 1-in. to 2-in. gravel, crushed stone and sand. Cement is received in bulk in box-cars, and is unloaded by power scraper to the hopper of a pneumatic conveying system whose 4-in. pipe leads to a large steel bin beside the aggregates bin. Compartments in this bin enable the use of three different brands of cement to meet varying specifications. Aggregates discharge from the bin compartments to a three-beam weighing hopper with dial index. Cement discharges similarly in a weighing batcher with a two-beam scale and dial. Water is metered into an open-top tank. Aggregate, cement and water enter a 3-cu. yd. mixer and the mixing is timed by a batchmeter set at 2 min. 10 sec. Allowing for closure of all gates, this assures a full 2-min. mix. A spotlight plays in the mixer to permit the operator clear view. Concrete is delivered by trucks with agitator bodies. A high-lift hoist assures a quick discharge. Average haul is 3½-mi. All plant machinery is electrically driven.—A. J. HOSKIN

Lac Seul conservation dam, Ear Falls. T. H. HOGG. *Can. Eng.*, Nov. 5, 1929, V. 57, No. 19, p. 684.—Concrete dam of concrete piers and deck with 20 stop log sluices, built in 1928 on the English River below the outlet of Lac Seul by the Ontario and Manitoba governments. Maximum height of dam is 40 ft. Main piers 7 ft. wide for deep section and 5 ft. for shallower. Deck is of reinforced concrete 26 ft. wide.—C. R. YOUNG

New underground railway offices, St. James Park. Notes on foundations and erection. M. J. MCCARTHY. *Structural Engineer*, (England) Nov. 1929, V. 7, No. 11, p. 403-418.—Foundations of new building of the Underground Railway Headquarters in London were featured, among other things, by the use of 700 Vibro cast-in-place concrete piles averaging about 30 ft. long. The method and order of driving are given together with the method used in computing the estimated ultimate driving resistance of individual piles, the computations involving the use of a driving resistance curve. The specified maximum safe load for each pile was 50 tons with a margin of safety between 3 and 4. Concrete for the piles was of 1:2:4 mix for which Ferrocrete, the rapid-hardening p. c. was used. Details of the capping for supporting loads of 250 and 800 tons are shown. Two guy derricks were used to considerable advantage in erecting the steel frame work of the building. A single Insley mast plant was used for placing concrete in walls and floors. A mixer of ½ yd. capacity, on the ground level, discharged into a bucket which was then hoisted and automatically discharged upon reaching the head frame into a 30-ft. chute. Thence from the chute the concrete was led into receiving hoppers of 30 cu. ft. capacity on the various floors. Concrete was discharged into 6-cu. ft. two-wheeled handcarts for distribution.—V. P. JENSEN

Mixer truck for premix concrete and concrete aggregates. *Rock Products*, Nov. 9, 1929, V. 32, No. 23, p. 106.—Stepanian mixer for mixing aggregates and water in transit or at destination or for carrying premix concrete described. Equipment includes belt conveyor and chute, adjustable to discharge at different heights.—J. J. LANDY

Metropolitan Boston's first ready-mixed concrete plant. M. N. CLAIR. *Rock Products*, Nov. 9, 1929, V. 32, No. 23, p. 107—Details of design and operation; description of equipment; printed forms for deliveries and records; and schedule of prices. Quality control plan given and some unusual data on special batch-mixing.—J. J. LANDY

The Olympia extensions. Visit of the members of the institution. *Structural Engineer*, (England) Nov. 1929, V. 7, No. 11, p. 427-430.—Joseph Emberton, architect of the new building under construction, was permitted to exercise his views concerning simplicity and truthfulness in architectural treatment. Since speed is the essence of the work, a number of engineering firms have been carrying out the fabrication. A record for the country was sent by the erection of 5000 tons of steel in 10 weeks. Main columns are made up of triple I-beams with wide flange plates and having heavy flats riveted to the webs. Girders are built-up plates and angles. Hollow reinforced concrete beams span between the steel joists.—V. P. JENSEN

The Denham-Rickmansworth main road. *Concrete Constr. Eng.*, (England) Nov. 1929, V. 24, No. 11, p. 653-658.—Construction was begun near Denham in May, 1927. The road is 2¼ miles long and 30 ft. wide between curbs. A foundation of 4 in. of clinker and 2 in. of gravel was used under the 8-in. two-course plain concrete slab. The bottom course consists of 6½ in. of 1:2:4 concrete, made with rapid hardening portland cement and 2-in. coarse aggregate. The second course was 1½ in. of 1:1 ½:3 concrete, made with ¾-in. granite coarse aggregate. The curbs were precast, laid on a plain concrete bed. Longitudinal tongue and groove joints were used, also transverse joints. For the lower course the materials were transported to the mixer by industrial railway. The mixer, of 12-8 cu. ft. capacity, was mounted on a track and pulled along the subgrade by means of a gasoline winch. The mixer discharged into a side dumping bucket mounted on rails which extended across the half-width of road being poured. The average rate of concreting was 400 sq. yds. per 8-hr. day; the maximum, 600. Seventy men were employed on the work. Hand tamping and finishing methods were used. At the end of a day's work, the concrete was covered with canvas, kept wet until morning. The surface was then covered with sand. Before opening to traffic it was treated with sodium silicate.—F. E. RICHART

Foundations for flats at Westminster. *Concrete Constr. Eng.* (England), Nov. 1929, V. 24, No. 11, p. 639-41.—Pile foundations extending through loose fill into blue clay were used in 5-story flat buildings at Westminster. Four hundred thirty-three "Simplex" reinforced concrete piles, 16 in. in diameter and averaging 34 ft. 6 in. long, were used. The building walls were supported on reinforced concrete beams spanning between the piles.—F. E. RICHART

Reinforced concrete reservoir at Barnet. *Concrete Constr. Eng.* (England), Nov. 1929, V. 24, No. 11, p. 643-6.—The new 5-million gallon reservoir for Barnet district is a combination of reinforced and mass concrete. It is nearly square, 290 ft. long and from 236 to 266 ft. wide, with provision for 12-ft. depth of water. The roof slabs, beams, columns, floor and wall foundations are reinforced; the walls, which vary in thickness from 7 ft. at bottom to 2½ ft. at top are 1:3:6 mass concrete. Rapid hardening cement was used and Blawforms were employed in pouring the walls.—F. E. RICHART

Sewage disposal tanks at Luton. *Concrete Constr. Eng.* (England), Nov. 1929, V. 24, No. 11, p. 647-651.—Six new settling tanks, 50 ft. square, have been added to the sewage disposal works of the Borough of Luton. The upper part of each tank is of reinforced concrete, the vertical walls being 8 in. thick. The pyramidal bottoms of the

tanks, resting on chalk, are not reinforced. Reinforced baffle walls, 13 ft. square and 4 in. thick are used in each tank. Metal forms were used in most of the work. The concrete was of proportions 1:6, using rapid hardening cement. Reinforcement was of mild steel. The concrete was poured in 6 ft. lifts.—F. E. RICHART

Construction of the James River bridge project. R. C. WILSON AND HERBEPT B. POPE, *Proc. Am. Soc. Civil Engr.*, Nov. 1929, V. 55, No. 9, p. 2357-74.—This $6,000,000 toll project, built in 1928, consisting of three bridges totaling 5 2/3 miles in length and 11 miles of concrete approach highways, eliminates detours and ferries for north and south travel through Norfolk, Va. The largest bridge, crossing the James River near Newport News, is 4½ miles long. The unique features were the casting, handling and driving of large concrete piles and load tests on piles which did not drive to refusal. Largest piles were 115 ft. long, 24 in. square and weighed over 35 tons each. With a penetration of 90 ft. of soft clay many of them did not drive to refusal and tests were made on these. The gripping power of the clay became very great and piles that by penetration formulas were good for but 18 to 20 tons, actually carried 90 tons without settlement. Maximum bridge loads are 42 tons each. Estimated cost of $9.00 per ton of bridge load led to special design effort to keep the dead load to a minimum. Using reinforced concrete slabs on steel I-beams resulted in saving of 26,000 tons in 5 miles of trestle. Pile bents were spaced 44 ft. instead of 34 ft. on center. Saving was $300,000 on piles and piers and about as much on the deck due to simplicity of construction. From north shore ¾ mile of ascending typical trestle work leads to four 90-ft. plate girder spans followed by four 210-ft. through trusses. Channel span has lift bridge 300 ft. long with 50 ft. clearance closed and 145 ft. clearance fully opened. This exceeds East River (New York) clearances by 10 ft. Similar arrangement of trusses and plate girder spans lead to the southside trestle. Other trestle and bridge work further south includes roadway over swamp so soft that material was removed and sand was pumped in from river. Concrete piles and piers to Elevation + 5.0 were of 1:1½:3 mix and other concrete was 1:6. A modern concrete testing laboratory facilitated taking daily test cylinders which were broken at 7, 14 and 28 days. Other cylinders cured under various weather conditions were broken at 2 to 10 days, the early strengths being used to determine when forms and centering might be removed. An old World War shipyard made a satisfactory operating base in which the 2,980 concrete piles were cast and cured for 30 days. Casting yard capacity was 140 piles, the lengths being adjusted by putting end stoppers in forms. Longitudinal steel for piles was used in full lengths (117 ft. for the longest units, 115 ft. piles). Two 1-yd. mixers were fed by a 400-yd. charging hopper, served in turn by a stiff leg derrick charging from cars or stockpile. Aggregate was measured by batch hoppers and water content was accurately gauged. Cement came from the cement house by belt conveyor. A gantry crane lifted piles from forms after 7 days. After lifting, the pile was cured by water jets for 21 days or longer in the curing yard, after a thorough inspection. Curing yard capacity was 500 piles. Piles were driven by two large floating drivers of "single" and "four lead" types, the latter getting all 4 piles for a bent aligned before driving started. The long piles were handled by 3 or 4 point suspension to avoid excessive bending stresses. Two piles were driven at a time by two 7,500 lb. steam hammers. Some piles had to be cut off to grade with an acetylene torch and air hammers. Piles showing maximum penetration per blow were loaded with a water tank on an I-beam frame. Readings were taken for various loads up to 90 tons. Recoverance was measured after removal of load. The maximum added penetration under the 90 ton load was ⅛ in. Bridge concreting was done from 2 plants mounted on large barges. A bent consisted of a 9 in. slab on four 30-in. 115-lb. I-beam stringers, 43 ft. 9 in. long resting on

concrete caps supported by 4 piles each at 7 ft. on centers. Piers under trusses were in about 40 ft. of water and were supported on wood piles at river-bottom level. Two piers with bell shaped bottoms 23 ft. in diameter tapered to 10 ft. diameter web connected cylinders at the water surface. Steel forms were used.—H. J. GILKEY

Construction methods and plant layout at Coolidge dam in Arizona. J. G. TRIPP. *Proc. Am. Soc. Civil Eng.*, Nov. 1929, V. 55, No. 9, p. 2347-2356. —Construction of Coolidge Dam across Gila river, San Carlos Project, Arizona involved departures in plant layout. The dam is a symmetrical 3-dome structure with spillways at each end; in a box canyon, the river bed having nearly the same width as the crest of the dam. Height 245 ft., maximum thickness 212 ft. and concrete volume 204,000 cu. yd. The walls are 20 ft. thick at the bottom, tapered to 4 ft. at top. Span of the domes 94 ft. at bed rock measures to 160 ft. at midheight, decreasing to 126 ft. at top. At top dam recedes about 200 ft. from upstream foundation contact. Excavation amounted to 300,000 cu. yd. of rock and 50,000 cu. yd. sand, gravel and boulders. Concrete plant was on hillside instead of in the canyon, avoiding flood danger of Gila river. Aggregates stored ½ mile downstream. Aerial tramway transported material to the site in preference to a road or railroad in the canyon (subject to flood hazard), or a long serpentine up-grade along the side, on account of the lower operating cost. The "Construction Symmetry of the Design" controlled the final selection of the distributing system which was difficult because of the width of canyon at the base of the structure which made the lengths along the two axes nearly equal and the outline in plan rectangular rather than triangular as is usual for a dam in a V-canyon. The massive, lower portions of the dam were poured with a plant able to place 2,000 cu. yd. of concrete per day, while another plant was used for the higher, upper sections in some parts of which not over 100 cu. yd. per day could be placed. Operating economies justified the extra equipment necessary for the dual installation. The main hoisting tower on a concrete pedestal extending to bedrock and capable of withstanding a considerable flood, was just downstream from the crest line and in the middle of the canyon, in the river bed and extended to a height 375 ft. above it (about 130 ft. above the crest). Side-hill towers would have involved re-hoisting and high first cost. Gravity bins for aggregate at the west side of the canyon at about spillway level were filled by the aerial cableway and siloed cement used. Two 2-yd. mixers fed by batchers delivered the concrete to the base of the main tower by a chute. There the concrete was hoisted to a threeway chuting plant above, one each to east and west domes and central sections. The long chutes to east and west domes were each suspended from a cable one end of which was attached to the tower. The central chute was suspended directly from the tower by a boom that described a semi-circle about the tower. The ends of all chutes were fitted with swing counter-balances suspended from cables stretched across the canyon. The spandrel walls, domes, sidewalks, hand rails, enrichments, etc., were poured from automobile hopper car on a timber track.

The bins had an 8-hour storage capacity. The tramway operated 24 hours and concrete could be poured continuously through two 8 hour shifts. Mobility was necessary for the excavating plant because of the uncertain nature of the stream. Trucks of 7 to 10 ton capacity and 1¼ yd. shovels were adopted for the river excavation. The trucks could "track" forward or backward with equal ease. "Backdown" ramps were constructed into the excavation. For flood control and to protect the gravel plant from excessive water a 12 by 32 ft. adjustable gate was built to close against a hydrostatic head of 150 ft. Pumping units were of the vertical turbine type because of their general "fool proofness." They were suspended from cables stretched across the canyon and could easily be lifted away from harm by gasoline hoists. In addition to the

aerial tramway the general service plant included a suitable auxiliary layout including crushing and screening plant. The gravel pit material was a mixture of cobbles, boulders, silt and mud and had to be altered before using as graded aggregate. A dinky locomotive on narrow-gauge track transported gravel. Two inlet towers at the heel of the structure received material during construction with a 10-ton cableway which also carried forms and reinforcing steel to the lower areas of the domes. Most of the tonnage, however, was delivered over the 20-ton cableway centered above the roadway of the dam. Trucks shuttled up and down the roadway between camp, shops and bottom of the canyon. Electrical layout, air compressors and water service completed the plant.

In the construction leveling control was necessary for sloping "pours" in the buttresses. Domes were poured in three vertical lifts. Buttresses and quarter-spans of adjacent domes were poured in one piece. After 4 or 5 days for shrinkage the central part of the dome was poured. Flexible form panels, in assorted sizes were used throughout. The dome caps required some special manipulation in pouring. Ingenuity was required for some phases but it is concluded that the multiple dome was found to be a practical structure to build and a worthy contender for primacy in the high dam field.—H. J. GILKEY

The construction of the bridge near Echelsbach. FERD. DULL. *Zement*, (Germany) 1929, V. 18, p. 1305-8.—The construction of an arch-bridge in reinforced concrete under use of the system "Melan-Sachsenberg" is described. The arches were built from both sides without centerings. The iron structures were loaded with sand of the weight of the concrete to be used, before pouring the concrete, in order to balance all tensions.—A. E. BEITLICH

Continuous shoring speeds work on tall Chicago structure. C. W. ELLSWORTH AND H. S. KELLER. *Concrete*, Dec. 1929, V. 35, No. 6, p. 13.—The 201 North Wells St. Bldg., Chicago, covers the full size of the lot to the twenty-first floor, with a set-back at each floor to the 24th, which is the first floor in the tower proper. This, in turn, sets back at the corner of each floor to the 30th floor. From this point to what would be the 32nd floor, the roof is stepped back in the form of a pyramid. The framework around the elevators is structural steel up to the 21st. floor and from the 30th. to the apex of the pyramid. Throughout the first eight floors and in scattered portions of the upper floors, cast iron cores were used in the concrete columns. The locations of joist bottoms were determined from a typical floor plan and from the floor, locations of continuous shores could be spotted and projected down to the basement, taking care to miss all joists or other structural members. On the rough excavation of the basement floor, double mud sills were laid as a foundation for the 4 by 4 in. shores. These were run up above the elevation of the finished first floor. —C. BACHMANN

Past experiences with high dams and outlook for the future. A. J. WILEY. *Proc. Am. Soc. Civil Eng.*, Nov. 1929, V. 55, No. 9, p. 2318-26.—Masonry dams require hard and durable foundations, which is not necessarily true of an earth dam. The lateral thrust of an arch dam may be even more exacting on the side walls than on the base. Concrete masonry has largely replaced stone since 1910. In addition to the straight gravity type the following represent recent practice. (1) Arched gravity dam of long radius. (2) Various types of arch dams of shorter radius and less than gravity section. (3) Multiple-arch dams. (4) Buttress type or Ambursen dams. (5) Multiple dome dam. The Cheesman dam, 235 ft., and the Roosevelt dam, 280 ft., were among the last high arched gravity dams to be built of stone masonry and were also about the last to ignore uplift in their design. Current practice is to use deep cut-off trenches, pressure grouting and foundation drainage. Uplift in the body of the dam is guarded against by vertical drains near the upstream face. The high arched dams of less than gravity section are suitable for narrow canyon sites and may be of fixed or variable radius or variable radius with constant angle types. The multiple dome type is similar in adaptability to the multiple

arch but is more massive and avoids the relatively thin walls and buttresses of the latter. The Ambursen type is similar to the multiple arch but uses reinforced concrete slabs between buttresses. The author favors watertight construction to drainage for water that penetrates the concrete. The value of metal stops is questioned. Leakage occurs more frequently at the horizontal construction joints than it does at vertical construction or contraction joints. The value of the vertical drains is questionable. The use of sand cement is discouraged. Large-sized gravel or cobbles placed in the mixer is considered to effect a more desirable economy of cement. The value of pressure grouting as a means of increasing arch action is questioned. The author favors arch gravity sections and does not feel that contraction joints seriously reduce the added factor of safety obtained by arching. He further concludes: Masonry dams on suitable bedrock are safe and permanent structures and are most suitable for very high dams. The stability of an arched gravity dam is increased as the radius of curvature is shortened. If free from horizontal joints and seams the only uplift either in the concrete of the dam or between the dam and a well bonded base is that due to a difference in the degree of porosity of the concrete and foundation rock. In a homogeneous mass pressure in the pores of the material is self neutralizing.—H. J. GILKEY

Continuous truss bridge 1,575 ft. long at Cincinnati. *Eng. News-Record*, Nov. 7, 1929, V. 103, No. 19, p. 734.—Monolithic concrete piers were constructed on rock foundation overlaid with 40 to 50 ft. of silt sand and gravel, and offshore piers were supported on concrete piles with reinforced concrete sheet piles to protect footings. Existing south pier was widened by sinking caisson beside old pier, providing separate foundation. Extension was made to include a reinforced concrete jacket around old pier. New concrete was secured to old pier by dowels and steel channels held by tie bolts running through old pier. Concrete was spouted into forms from cars on tracks on existing bridge.—D. F. JENNINGS

Deep foundations for Lake Champlain bridge built with open cofferdams. *Eng. News-Record*, Nov. 21, 1929, V. 103, No. 21, p. 796-800.—Open cofferdams 97 ft. below lake level were sealed with concrete placed under water using bottom-dump buckets, in six-day continuous pours. After concrete in seal had hardened, cofferdams were pumped out and remainder of concrete was placed in the dry, using same automatic bottom-dump buckets. Buckets dumped automatically under water when in contact with the concrete; in the forms, by contact with cross bracing. Aggregates were iron mine tailings making heavy concrete. Proportions for under-water concrete were 1:1.8:3.6 with 6.5 gal. water per sack of cement and mixing time of 2 minutes; for shaft concrete, proportions were 1:2.5:5 with a W/C ratio of 7.7 gal. and 1½ min. mixing time. All concrete was handled by floating plant. Concrete bridge deck was placed in two strips, each one half width of deck, one half being completed before starting other half. Deck concrete was delivered in hopper cars handled on tracks on bridge.—D. F. JENNINGS

Why joints in concrete pavements. CLIFFORD OLDER. *Eng. News-Record*, Oct. 3, 1929, V. 19, No. 14, 530.—A theoretical discussion of the stresses set up in concrete slabs by temperature and moisture. Attention is called to the fact that spalling at joints and cracks may be due to excessive compressive stresses developed by expansion and warping of the pavement slab. Discusses types of contraction joints and defines function and proper design of dummy joints. With "low resistance" expansion joints, spalling as well as blow-ups should be largely if not completely eliminated by spacing such joints at least every 100 feet, provided intermediate contraction joints are installed at from 15 to 30 feet apart in order to control transverse cracking.—F. H. JACKSON

Establishing a state record in concrete road building. R. W. EDWARDS. *Roads and Streets*, Oct. 1929, p. 343.—Unusually rapid progress

was made in the construction of a concrete pavement in California through the use of efficient methods of management. Average production for an 8-hour day of 329 batches of concrete (27 E paver) using a 60 second mix, was maintained throughout the construction of 11.4 miles of 20-ft. pavement. Aggregates were batched by weight and two separate sizes of coarse aggregates employed.—F. H. JACKSON

Up-to-date concrete street pavement. E. C. SMITH. *Roads and Streets*, Nov. 1929, p. 417.—A summary of recent developments in the design and construction of concrete street pavements. Discusses importance of material control, measurement of aggregates by weight, etc. Describes tests used to determine time of opening streets to traffic. Includes a brief description of the design and construction practices in several cities.—F. H. JACKSON

Construction of concrete reservoirs with special reference to foundation work in abandoned mines. R. M. RIEGEL AND G. L. HENDRICKSON. *Proc. Eng. Soc. of West. Penn.*, Nov. 1929, V. 45, No. 8, 23 pages.—Foundations for two reservoirs were prepared over old coal workings which were beneath a 60-ft. sandstone roof. Crevices in settled rock complicated the problem, and coal stratum showed numerous cavities partly filled with gob consisting of fine coal, slate and shale. Experimental grouting was carried out in about a half dozen of the test-holes to develop a satisfactory method. A shaft was sunk adjoining one of the test-holes and some cavities were found to be filled with grout and some with a combination of mortar and gob. It was held to be demonstrated that the method so stiffened up the material adjacent to the drill holes that a columnar support was provided at this point capable of supporting the load contemplated. Holes were drilled over the area of both floor and supporting enbankment. These were packed just above the coal horizon and cement grout pumped in until a pressure of 100 lb. per sq. ft. was attained. Neat cement was used for the first few batches going into each hole and the mixture leaned to a 1:3 cement-sand grout, by volume. Packing was then loosened and pipes raised to the surface of the sandstone roof. Grout was then pumped in to seal off the rock crevices. Two Union Iron Works pneumatic grout machines with agitator paddles were used. Grout in one instance traveled 45 ft. into an adjacent stone quarry. The largest amount required by any one drill hole was 2775 cu. ft. and the total placed in 296 holes was approximately 140,000 cu. ft. The larger, or Brashear reservoir, was roughly oval in shape and was of T-wall type. The floor was of double slab construction with membrane waterproofing between. Footings were provided with a cut-off wall for seepage extending down to rock and drainage provided longitudinally with broken stone, outlets through the embankment being also provided. This was done to protect from both under-cutting and back-pressure. A feature was the use of copper-sealed joints between the construction units of the wall and footing and also between the footing and the adjoining slab of the floor. These joint strips were shaped on the job and were grooved to care for expansion. They were lapped, riveted and soldered at the ends and were mopped with asphalt as placed. Construction joints were subsequently poured with Trinidad asphalt. Because mix used was rather rich and some shrinkage cracks developed. Inside of the wall was given two coats of coal-tar waterproofing paint. The smaller, or McNaugher reservoir, was circular in shape with a smaller circular tank beside it to serve as pump regulator when the big tank might be out of service. Both were built with horizontal construction joints, insuring the monolithic character of the hoops. The tanks were waterproofed by guniting.—P. J. FREEMAN

Classification, selection and adaptation of high dams. (See ENGINEERING DESIGN.)

SHOP MANUFACTURE

Cold weather curing of concrete products improved by simple drying device. *Concrete*, Nov. 1929, V. 35, No. 5, p. 31.—A unit heater is similar

to an automobile radiator in appearance. It has a tier of copper steam tubes running in a horizontal direction. Around each copper tube a helical brass fin is wound which increases the heat radiating surface about 300%. The unit selected for use is about 30 in. square and about 10 in. thick, capable of delivering 50 sq. ft. of radiation. The heater was mounted on a portable chassis and the fan that had previously been used was mounted on the same chassis directly in front of the heater. The unit was then placed inside the curing tunnel directly against the door. The air supply was taken from the storage room through a 12-in. galvanized iron sleeve which extended from the fan housing through a hole in the tunnel door into the storage shed.—C. BACHMANN

Is this the way out? D. R. COLLINS. *Concrete*, Nov. 1929, V. 35, No. 5, p. 17.—A small group of Milwaukee County, Wisc., concrete products manufacturers began a series of meetings, culminating in a preliminary co-operative contract. Each plant was to be put on quota basis—to be established by an average of 1927 and 1928 business. When sufficient signatures had been secured, the incorporation of a non-stock co-operative association under the Wisconsin Marketing Act of 1927 was concluded. The association is organized without capital stock and any individual, partnership or corporation engaged in the manufacture of concrete products in Milwaukee County is eligible to membership upon signing a membership roll and upon acceptance by a majority of the board of directors. Each member is entitled to participate in profits in proportion to the volume of business contracted by members with the association.—C. BACHMANN

Handbook for the cast stone industry. *Das Betonwerk* (Germany) V. 17, No. 44.—A book of 865 pages profuse with diagrams and illustrations entitled "Handbuch der Zementwaren und Kunsteinindustrie." Methods, caststone, terrazzo, mosaic fully described.—H. FRAUENFELDER

Mold boxes. G. R. GREGORY. *Concrete Building* (England), Nov. 1929, V. 4, No. 11.—A description and drawings of six kinds of mold boxes used in the cast stone industry. Some ingenious methods of construction of molds are indicated.—H. FRAUENFELDER

Pre-cast concrete factory operation. *Concrete Building* (England), Nov. 1929, V. 2, No. 11. Concrete Publications, Ltd., 20 Dartmouth Street, Westminster, London, S. W. 1 offers book 216 pages 146 illustrations describing methods and processes used by more than 20 English manufacturers of all classes of precast concrete.—H. FRAUENFELDER

The bonding of Rhenish pumice stones with lime and cement. (See MATERIALS.)

Manufacture of art objects from colored cement. GEORGE RICE. *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 121.—Part II of the series dealing with making majolica tile as inserts in metal structures.—J. J. LANDY

Limestone screenings in concrete products. L. C. HILL *Rock Products*, Nov. 23, 1929, V. 32, No. 24, p. 122-123.—Comparative tests on all-sand concrete block and block made with limestone-sand aggregate indicate. Latter block has higher compressive strength and lower absorption rate. Screen analysis of different mixes given. Leaner mixes improved by addition of limestone screenings. Ideal fineness modulus for concrete products mix is between 3.75 and 4.25, 25 per cent being retained on No. 4 sieve.—J. J. LANDY